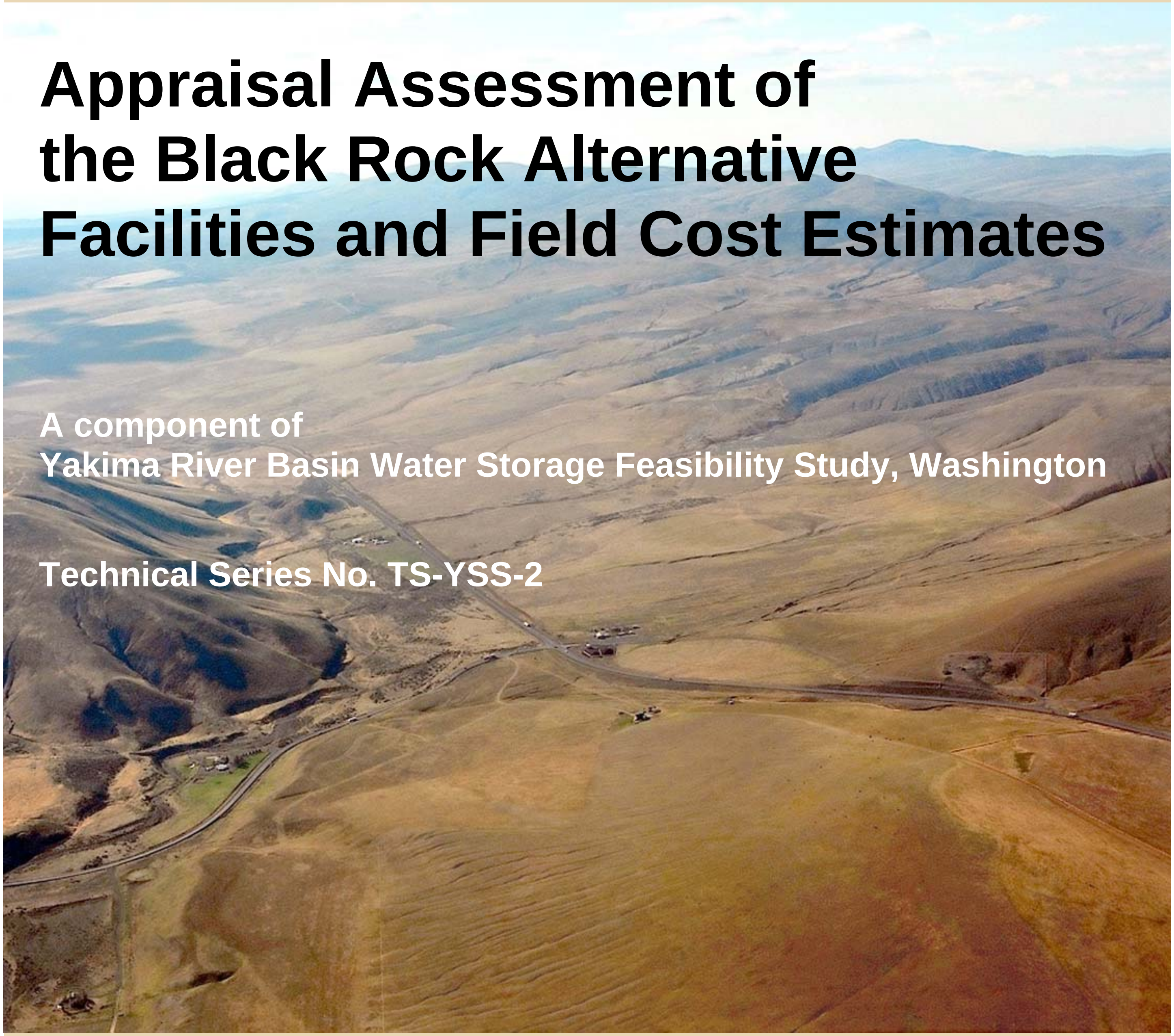
RECLAMATION
Managing Water in the West

Appraisal Assessment of the Black Rock Alternative Facilities and Field Cost Estimates

A component of Yakima River Basin Water Storage Feasibility Study, Washington

Technical Series No. TS-YSS-2



Black Rock Valley

U.S. Department of the Interior
Bureau of Reclamation
Technical Service Center
Denver, Colorado

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U.S. Department of the Interior

Mission Statement

The Mission of the U.S. Department of the Interior is to protect and provide access to our Nation's natural and cultural heritage and honor our trust responsibilities to Indian tribes and our commitments to island communities.

Mission of the Bureau of Reclamation

The Mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.

Preface

Congress directed the Secretary of the Interior, acting through the Bureau of Reclamation (Reclamation), to conduct a feasibility study of options for additional water storage for the Yakima River basin. Section 214 of the Act of February 20, 2003, (Public Law 108-7) contains this authorization and includes the provision "… with emphasis on the feasibility of storage of Columbia River water in the potential Black Rock Reservoir and the benefit of additional storage to endangered and threatened fish, irrigated agriculture, and municipal water supply."

Reclamation initiated the *Yakima River Basin Water Storage Feasibility Study* (Storage Study) in May 2003. As guided by the authorization, the purpose of the Storage Study is to identify and examine the viability and acceptability of alternate projects by: (1) diversion of Columbia River water to the potential Black Rock reservoir for further water transfer to irrigation entities in the lower Yakima River basin as an exchange supply, thereby reducing irrigation demand on Yakima River water and improving Yakima Project stored water supplies, and (2) creation of additional storage within the Yakima River basin. In considering the benefits to be achieved, study objectives will be to modify Yakima Project flow management operations to most closely mimic the historic flow regime of a Yakima River system for fisheries, provide a more reliable supply for existing proratable water users, and provide additional supplies for future municipal demands.

State support for the Storage Study was provided in the 2003 Legislative session. The capital budget included a $4 million appropriation for the Department of Ecology (Ecology) with the provision the funds "… are provided solely for expenditure under a contract between the department of ecology and the United States bureau of reclamation for the development of plans, engineering, and financing reports and other preconstruction activities associated with the development of water storage projects in the Yakima river basin, consistent with the Yakima river basin water enhancement project, P.L. 103-434. The initial water storage feasibility study shall be for the Black Rock reservoir project."

Reclamation's Upper Columbia Area Office in Yakima, Washington, is managing and directing the Storage Study. Pursuant to the legislative directives, Reclamation has placed initial emphasis on Black Rock alternative study activities. These study activities are collectively referred to as the Black Rock Alternative Assessment (Assessment).

The Assessment has three primary objectives. First, it provides the emphasis directed by Federal and State legislation. Second, it builds upon prior work and studies to provide more information on the configuration and field construction cost of the primary components of a Black Rock alternative. It examines legal and institutional considerations of water supply and use, and identifies areas where further study is needed. Third, it is a step forward in identifying the viability of a Black Rock alternative.

This technical document, prepared by Reclamation's Technical Service Center, Denver, Colorado, is one of a series of documents prepared under the Storage Study. This particular document is a component of the Assessment reporting on preliminary appraisal-level engineering evaluation of designs and field cost estimates of potential Black Rock alternative facilities to withdraw, store, and convey Columbia River water to irrigation entities in the lower Yakima River basin. Information and findings of this technical document are included in the Assessment Summary Report.

Further Consultations

The information available at this time is necessarily preliminary, has been developed only to an appraisal level of detail, and is therefore subject to change if this alternative is investigated further in the course of the Yakima River Basin Storage Feasibility Study (Storage Study). Finally, economic, financial, environmental, cultural, and social evaluations of the Black Rock alternative have not yet been conducted.

The policy of the Bureau of Reclamation (Reclamation) requires non-Federal parties to share the costs of financing feasibility studies and the eventual construction of Federal reclamation projects. In light of this policy, the preliminary cost estimates presented in the Assessment Summary Report, and current Federal budgetary constraints, Reclamation is not reaching a decision at this time as to whether the Black Rock alternative will be carried forward into the next phase of the Storage Study or dropped from further consideration. Rather, Reclamation will consult with the State of Washington (which is cost sharing in the Storage Study), the Yakama Nation, the potential water exchange participants, project proponents, and other interested parties before making a decision in this regard. It is anticipated that a decision will be reached by the fall of 2005.

If the Congress provides further funding for the Storage Study, all technically viable alternatives would be compared and an alternative(s) selected for further analyses in the feasibility phase. (Whether the Columbia River-Yakima River water exchange concept in the form of the Black Rock alternative is included will depend upon whether Reclamation, after these additional consultations, decides to carry that alternative forward into the plan formulation phase of the Storage Study.) The selected alternative(s) would then be subject to detailed evaluation in the feasibility phase in terms of engineering, economic, and environmental considerations, and cultural and social acceptability. This feasibility phase would be the last phase of the Storage Study. Preparation of the Feasibility Report/Environmental Impact Statement would be a part of this final phase.

Appraisal Assessment of the Black Rock Alternative Facilities and Field Cost Estimates

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List of Abbreviations and Acronyms

af	Acre-feet
cfs	Flow rate in cubic feet per second
El.	Elevation
fps	Velocity in feet per second
ft	Foot or feet
ft^2	Area in square feet
ft^3	Volume in cubic feet
g	Acceleration of gravity (32.2 ft/s^2)
HGL	Hydraulic Grade Line
hp	Horsepower
H:V	Ratio of horizontal to vertical slope
ID	Inside diameter
kV	Kilovolt
kVA	Kilovolt-amperes
kwh	Kilowatt hours
lbs	Pounds
lf	Linear feet
miles/hr	Miles per hour
mm	Millimeter
MP	Mile post
MW	Megawatt
NMFS	National Marine Fisheries Service of the National Oceanic and Atmospheric Administration
OD	Outside diameter
PHA	Peak Horizontal Acceleration
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
PSHA	Probabilistic Seismic Hazard Assessment
psi	Pressure in pounds per square inch
Q	Flow rate
RCC	Roller Compacted Concrete
rpm	Revolutions per minute
SH	State Highway

List of Abbreviations and Acronyms
(continued)

TSC	Technical Service Center
USGS	United States Geologic Survey
WIS	Washington Infrastructure Services, Inc.
WR^2	Pump Moment of Inertia
°	Degree
%	Percent

Related Reclamation Documents

- Preliminary Appraisal Assessment of Columbia River Water Availability for a Potential Black Rock Project, Technical Series No. TS-YSS-1, Prepared by Pacific Northwest Regional Office, 2004.

- Preliminary Assessment of Black Rock Delivery System for Roza, Terrace Heights, Selah-Moxee, and Union Gap Irrigation Districts, Technical Series No. TS-YSS-3, Prepared by Pacific Northwest Construction Office, 2004.

- Preliminary Assessment of Black Rock Delivery System for Sunnyside Division, Technical Series No. TS-YSS-4, Prepared by Pacific Northwest Regional Office, 2004.

- Preliminary Assessment of Geology at Black Rock Damsite, Technical Series No. TS-YSS-5, Prepared by Pacific Northwest Regional Office, 2004.

- Preliminary Assessment of Hydrogeology at Black Rock Damsite, Technical Series No. TS-YSS-6, Prepared by Pacific Northwest Regional Office, 2004.

- Summary Report – Appraisal Assessment of Black Rock Project, Technical Series No. TS-YSS-7, Prepared by Pacific Northwest Regional Office, 2004.

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Appraisal Assessment of the Black Rock Alternative Facilities and Field Cost Estimates

Technical Findings and Conclusions

The objective of the Black Rock Project is to deliver Columbia River water to Yakima Project entities susceptible of receiving such water, and willing to exchange it for all or part of their current Yakima River diversions. Currently, these exchange participants consist of the Roza and Sunnyside Irrigation Districts who have expressed a willingness to consider water exchanges. In the future, it may also be possible to exchange water with other entities such as the Union Gap Irrigation District, Selah-Moxee Irrigation District, and the Terrace Heights Irrigation District. This report documents an appraisal assessment of likely configurations, sizes, and costs of Black Rock Project facilities needed to pump, store, and deliver water to willing exchange participants. It will be used to better define the project and/or project components to be carried into detailed feasibility analysis.

Three main options were considered during this study.

Option 1: The Large Reservoir - Pump Only Option includes an intake with fish screens from Priest Rapids Reservoir, a 3,500 cfs pumping plant to lift the water to Black Rock Valley, a dam to store 1,300,000 acre-feet of active storage in Black Rock Reservoir, and a 2,500 cfs outflow tunnel and pipeline from the reservoir to Roza Canal.

Option 2: The Large Reservoir - Pump-Generating Option is similar to the Large Reservoir - Pump Only Option except it also includes a multi-level intake to selectively withdraw water from Black Rock Reservoir back to the Columbia River, a 3,500 cfs powerplant, and a 3,500 cfs tailrace channel to return the water back to Priest Rapids Reservoir.

Option 3: The Small Reservoir - Pump Only Option includes an intake with fish screens from Priest Rapids Reservoir, a 6,000 cfs pumping plant to lift the water to Black Rock Valley, a dam to store 800,000 acre-feet of active storage in Black Rock Reservoir, and a 2,500 cfs outflow tunnel and pipeline from the reservoir to Roza Canal.

The following conclusions are based on the technical and cost analyses completed for this assessment study:

1. Construction of facilities to pump, store, and deliver Columbia River water to willing exchange participants in the Yakima Basin is technically viable.

2. Appraisal-level field cost estimates for constructing facilities to pump, store, and deliver Columbia River water to the Roza Canal range from $2.46 billion to $2.65 billion (June 2004 price levels). Field cost estimates include costs for the principal items of work, mobilization costs, and allowances for unlisted items and contingencies. Field cost estimates do not include non-contract costs.

3. The appraisal-level field cost estimates for the Large Reservoir – 3,500 cfs Pump Only Option (Option 1) and the Small Reservoir – 6,000 cfs Pump Only Option (Option 3) are the same. Both reservoir sizes should be considered during future feasibility studies. Further analysis of the extent of the water exchange, timing of Columbia River water availability and diversions, economics, and other aspects will help refine the most desirable Storage-Pump Option.

4. The appraisal-level field cost estimate for the Large Reservoir – 3,500 cfs Pump-Generating Option (Option 2) is $190 million greater than the field cost estimate for the Large Reservoir – 3,500 cfs Pump Only Option (Option 1). However, operational studies have not been completed for the Pump-Generating Option and these studies may indicate a need to increase plant capacity to ensure annual delivery of exchange water.

5. The appraisal-level field cost estimate for the All Tunnel (Discharge 1) inflow conveyance system is significantly less than the cost estimate for the Tunnel/Pipe (Discharge 2) inflow conveyance system. The Tunnel/Pipe alternative should be removed from further evaluation.

6. The appraisal-level field cost estimates for the Black Rock embankment dams are significantly lower than the cost estimates for the roller compacted concrete (RCC) dams. The RCC dams should be removed from further evaluation.

7. There is not a significant cost difference between the concrete face rockfill and central core rockfill dams. Both of these embankment dams should be considered during future feasibility studies.

8. The difference between the appraisal-level field cost estimates for the 1,500 cfs and 900 cfs Black Rock Powerplants at the Roza Canal is small (less than 2 percent). The majority of the field cost is associated with the bypass structure that was assumed to have the same capacity (2,500 cfs) for each plant. The selection of which option to pursue should consider costs associated with the Roza and Sunnyside Delivery Systems.

9. The appraisal-level field cost estimate for the Sunnyside Powerplant and Bypass Structure located at the end of the Sunnyside Delivery System is $47.0 million (June 2004 price levels).

Level of Study

This technical document provides the results of an appraisal-level engineering evaluation of the primary components of the proposed Black Rock Project. This study is identified as Objective 301.4.2.A/Task 1 of the Yakima River Basin Water Storage Options Feasibility Study, Plan of Study [1]. The purpose of this evaluation is to develop and screen options to be considered during future detailed feasibility investigations, and to bring preliminary designs of Black Rock Project facilities to the same level of detail as other identified storage options in the Yakima Basin. The Assessment Study's focus was to develop a better definition of features, understanding of project constraints, and more accurate construction cost estimates for features required to transfer water from the Columbia River to the Yakima Basin via a new Black Rock Reservoir.

This study is based on available existing design data from past work accomplished by Washington Infrastructure Services, Inc. (WIS) and the Bureau of Reclamation (Reclamation), and is generally limited to the references listed at the end of this report. Aerial topography developed by Reclamation and limited geologic explorations conducted near the proposed damsites were also used to better define features. The amount of data collection is not considered to be at the level required for feasibility-level assessment of project features. Design data collected for future studies can cause future cost estimates to significantly deviate from the cost estimates presented in this report.

Options developed in this study have not been subject to detailed design and value engineering. Preliminary identification and sizing of required features was accomplished based on engineering judgment, limited analyses and available design data. Field cost estimates prepared for this study were generated using industry-wide accepted cost estimating methodology, standards, and practices. Major features were broken down into pay items and approximate quantities were calculated for these items based on preliminary general designs and drawings. Unit prices, adjusted for location and current construction cost trends, were determined for the identified pay items.

The appraisal-level field cost estimates developed for this Assessment are for the sole purpose of screening potential facility options and developing preliminary configurations of the Black Rock alternative. **The cost estimates in this report are not intended to be at the feasibility-level required to request project authorization for construction and construction appropriations by Congress.**

Appraisal Assessment of the Black Rock Alternative Facilities and Field Cost Estimates

I. Introduction

The Black Rock Project is one of the options to be considered under the Yakima River Basin Water Storage Feasibility Study. Legislation authorizing this study requests Reclamation to conduct a feasibility study of options for additional water storage in the Yakima River Basin, Washington, with emphasis on the feasibility of storing Columbia River water in the potential offstream Black Rock Reservoir. The objective of the Black Rock Project is to deliver Columbia River water to Yakima Project entities susceptible of receiving such water, and willing to exchange it for all or part of their current Yakima River diversions. Currently, these exchange participants consist of the Roza and Sunnyside Irrigation Districts who have expressed a willingness to consider water exchanges. In the future, it may also be possible to exchange water with other entities such as the Union Gap Irrigation District, Selah-Moxee Irrigation District, and the Terrace Heights Irrigation District.

This Appraisal Assessment Study is identified as Objective 301.4.2.A/Task 1 of the Yakima River Basin Water Storage Options Feasibility Study, Plan of Study [1] and was requested to be performed by the Denver Technical Service Center (TSC) by the Upper Columbia Area Office (UCAO) of the Bureau of Reclamation's Pacific Northwest Region. Additional engineering work will be accomplished during the future Feasibility Study identified as Objective 401.1.1/Task 1 of the Plan of Study.

II. Purpose of Engineering Work

Under contract with the Benton County Sustainable Development Department, Washington Infrastructure Services, Inc. (WIS) completed a reconnaissance-level analysis to identify and compare multiple options to transfer water from the Columbia River to the Yakima Basin. The results of their study are documented in the Black Rock Reservoir Study - Final Report, dated May 2002 [2]. Cost estimates developed for the WIS study were used to compare options against each other and develop an order-of-magnitude estimate of project costs however; detailed design and cost analysis of any one option were not completed. Reclamation's Assessment Study used the WIS Report and data obtained since the report was completed to develop a few options in greater detail to permit a better definition of required features, understanding of project constraints, and development of more accurate construction cost estimates; and to use these cost estimates to compare options. Reclamation's Assessment also developed features and cost estimates required for delivery of Black Rock water to the Roza and Sunnyside Irrigation Districts which were not included in the WIS Report. Details of these features are described in separate reports [3] [4].

This report documents an appraisal-level assessment of likely configurations, sizes, and costs of Black Rock Project facilities to pump, store, and deliver water to willing exchange participants. It will be used to better define the project and/or project components to be carried into detailed feasibility analysis.

III. Basis of Designs

Existing Conditions

The Yakima River Basin is located in south-central Washington. As part of this Assessment, a Project Site Review Team was formed to review existing data and evaluate potential sites for features associated with the Black Rock Project. The Site Review Team visited the project area on October 23-27, 2003. Major findings and discussions are documented in a Travel Report that is included in Appendix A.

Water Supply and Needs

The availability of Columbia River water in excess of instream target flows for exchange with willing Yakima River Basin water users was investigated by Reclamation prior to the sizing of the features for this Assessment. The results of the water availability study are documented in the Preliminary Appraisal Assessment of Columbia River Water Availability for a Potential Black Rock Project Report [5]. The findings of the water availability study with direct impacts on this study are listed below and summarized in Table 1.

- Columbia River water appears to be available for exchange with willing Yakima River Basin water users contingent on obtaining State authorization in some form of water right approval.

• Instream flow targets at various points on the Columbia River downstream from Priest Rapids Dam limit diversions in every month except September, and the October flow target is relatively low.

• Because of the timing of water availability in excess of instream flow targets and Columbia River water supply deficiencies in some dry years, direct delivery (without storage) during the irrigation season to the Roza and Sunnyside Irrigation Districts is not viable. A Black Rock reservoir would be required in order to affect a water exchange with the Roza and Sunnyside Irrigation Districts.

• For a 1,300,000 acre-foot active capacity reservoir and a 3,500 cfs pumping plant, water diversions from the Columbia River would have to occur throughout the year during both light and heavy electric load hours to meet the water delivery criteria over an extended period. Diversions would only occur when water in excess of instream target flows is available and there is reservoir capacity available to store water.

• For an 800,000-acre-foot active capacity reservoir and a 6,000 cfs pumping plant, water diversions from the Columbia River would have to occur throughout the year during both light and heavy electric load hours to meet the water delivery criteria over an extended period. Diversions would only occur when water in excess of instream target flows is available and there is reservoir capacity available to store water.

Table 1. Summary of Data from Water Availability Study

		Large Reservoir	Small Reservoir
Active reservoir capacity		1,300,000 af	800,000 af
Water exchange April - October wet and average years		810,400 af	810,400 af
Water exchange April - October that meets water exchange delivery criteria in Yakima River basin dry years		662,000 af [a]	662,000 af [a]
Assumed seepage loss		15,000 af	15,000 af
Assumed evaporation loss		30,100 af	23,470 af
Months to fill		6 to 30 months	2 to 13 months
Average August end-of-month reservoir content	(Active)	888,000 af	468,000 af
Average August end-of-month reservoir elevation		El. 1721	El. 1646
Average August reservoir percent full		68%	59%
Pump capacity that meets water exchange delivery criteria			
Heavy and light load hours pumping		3,500 cfs	6,000 cfs
Light load hours pumping only		9,000 cfs	15,500 cfs

[a] The water delivery criteria is the sum for the Roza and Sunnyside Divisions of all authorized nonproratable water and: 100 percent of the nonproratable water in wet and normal water years and a minimum of 70 percent of proratable water in Yakima River basin dry years.

Topography

This Assessment utilized aerial photogrammetry that was developed for the Yakima River Basin Water Storage Feasibility Study. Survey control for the aerial flight was installed under the direction of Reclamation's Ephrata Survey Crew and the aerial flight and photogrammetric process were done by Aerometrics, Inc. in August 2003. Grids, contours, and orthophotos were generated from the resultant data by the Technical Service Center. The flight was done at approximately a 1:10,000 scale (5,000 feet above the ground surface). This enables plus/minus 0.5 foot accuracy in elevation and slightly better in the horizontal. Two foot contours were generated for most of the design work and accuracy was within mapping standards. The aerial photogrammetry covers the Columbia River intake area, Black Rock Reservoir area, Black Rock Outlet area, and most areas in between. For locations outside the coverage area, including the delivery systems for Roza and Sunnyside Irrigation Districts, and a small portion of the Outflow Conveyance System between Black Rock Reservoir and Roza Canal, 7.5 minute USGS maps with 20-foot contour intervals were used to determine topographic information.

Geology

The Black Rock Damsite was initially studied by WIS in 2002 after the completion of their reconnaissance study. WIS field investigations involved drilling five test borings and excavating ten test pits in the vicinity of their preferred dam alignment. The results of this investigation are documented in the Black Rock Reservoir Study -Initial Geotechnical Investigation - Final Report [6]. This geotechnical investigation indicated that the depth of overburden along the preferred (original) dam alignment was much greater (up to 200 feet) than had been assumed in WIS's reconnaissance study (20 feet). Based on this field exploration and geologic mapping completed during the geotechnical investigation, an alternate dam alignment located further west was hypothesized to be less complicated due primarily to the potential presence of a north-south fault that was believed to place the bedrock nearer the ground surface.

Reclamation performed field investigations of this alternate dam alignment between December 2003 and June 2004. These investigations involved drilling five shallow holes to define the bedrock surface, one deep hole to confirm the stratigraphy of the deep foundation, and one deep hole for hydraulic conductivity testing. The drilling information showed the depth to bedrock and overburden thickness at the alternate site was actually greater than the original damsite, indicating that if a north-south fault exists between the sites, the offset is insignificant. Reclamation geologic investigations will be documented in the Preliminary Assessment of

Geology at Black Rock Damsite Report [7] . The following is a brief description of the geology associated with the project area.

Regional Geology

The Black Rock Dam and Reservoir sites are located in the northwest-central portion of the Columbia Basin, a structural and depositional basin that forms much of eastern Washington. The basin is the site of large basalt flood lava known as the Columbia River Basalt Province. The basalts were erupted between 18 and 6 million years ago from vents near the present boundary between Washington, Oregon, and Idaho. Flows were up to 100 feet thick and cover hundreds to thousands of square miles. Extended time periods between eruptions allowed for sediment deposition. Sediments consisted primarily of lacustrine silt and fluvial sand and gravel. Basalt eruptions over millions of years resulted in a stack of relatively horizontal flows which are referred to as the Columbia Plateau.

Structural Geology

Shortly after the onset of the eruptive activity the western portion of the Columbia Plateau underwent north-south compression resulting in east-west and northwest-southwest trending folds. These folds are referred to as the Yakima Fold Belt. The ridges of the Yakima Fold Belt are generally asymmetrical, with one limb gently inclined while the other steeply folded, often with a thrust fault near the base of the fold. The anticlines represent the ridges and the synclines represent the valleys. This configuration exists at the Black Rock Damsite, which is between the Yakima Ridge anticline on the north, and Horse Thief Mountain/Rattlesnake Hills anticline on the south, and similarly at the Priest Rapids Intake, Pumping Plant and Inflow Conveyance System sites, which are bounded by or within the Umtanum anticline.

Black Rock Damsite Geology

The Black Rock damsite is underlain by an interbedded sequence of volcanic and sedimentary rocks of the Columbia River Basalt Group and late Pliocene to Holocene age fluvial, lacustrine and wind-blown deposits. The upper foundation at the damsite is composed of quaternary loess and alluvium deposits underlain by sedimentary Tertiary Ringold Formation. The deep foundation bedrock is composed of volcanic rocks of the Saddle Mountain Basalt and upper Wanapum Basalt formations of the Columbia River Basalt Group.

The alluvial units documented at the damsite include pediment and alluvium deposits. These are underlain by the Ringold Formation which consists of moderately- to well-indurated fine- to coarse-grained sediments deposited within the Yakima Fold Belt. The underlying Columbia River Basalt and Ellensburg Formation sedimentary interbeds include the following: the Rattlesnake Ridge sedimentary interbed, Pomona basalt member, Selah sedimentary interbed, Esquatzel and Umatilla Basalt members, and the Mabton sedimentary interbed. The upper Priest Rapids Basalt of the Wanapum Basalt formation was encountered during exploratory drilling.

Landslides frequently are present in the Yakima Fold Belt. The slides form on the sloping limbs of the anticlines due to failure of the lower strength sedimentary interbeds. Two ancient landslides have been identified on the Horse Thief Mountain anticline that forms the south (right) abutment of the dam. The first slide is located on the north limb of the anticline upstream of the damsite; the second is downstream of the damsite on the plunging east slope of the anticline.

Groundwater in the Black Rock Valley occurs primarily in the basalt interflow zones which generally include a flow breccia at the bottom of one flow and the vesicular flow top of the underlying flow. The dense interior section of the basalt flows are confining layers between the interflow zones. Where the interbedded sediments are coarse-grained, the interbeds are included in the aquifer; however, the interbedded sediments are often fine grained and act as confining layers. During Reclamation's geologic investigations a deep hole was drilled to better define groundwater in the Black Rock Valley. Groundwater in the hole was first encountered during drilling at about 254 feet, at the bottom of the Pomona Basalt. The water level rose in the hole and the static water level was about 195-feet below ground surface [8].

Priest Rapids Intake, Pumping Plant, and Inflow Conveyance Geology

The geologic conditions for the intake, pumping plant and inflow conveyance structures are based primarily on information provided by the Grant County Public Utility District that was used to design and construct Priest Rapids Dam. The pumping plant and intake structures adjacent to Priest Rapids Reservoir will likely be founded on either Priest Rapids Basalt or Columbia River terrace deposits. The inflow tunnel will penetrate the Umtanum and Yakima anticlines which are composed of folded and faulted Saddle Mountains Basalt, Wanapum Basalt and possibly Grande Ronde Basalt formations. In the Priest Rapids Dam area, the north limb of the Umtanum anticline is overturned and dips to the south. An upper fault, the

Buck thrust, and a lower fault, Umtanum fault, define the overturned flows of the fold. Landslides have been identified along the steep overturned north slope of the anticline. Groundwater in the vicinity will be influenced by water levels in Priest Rapids Reservoir and Columbia River. Based on limited permeability and exploration data, it appears that the pumping plant will be located in an area with a shallow thickness (less than 20 feet) of unconsolidated terrace gravel lying above the Priest Rapids basalt and it is expected that the excavation for the pumping plant will have relatively minor water control needs.

Seismic Hazard

An initial probabilistic seismic hazard assessment (PSHA) was conducted for use in this Assessment Study of the proposed Black Rock Dam and is included as Appendix B. This PSHA is based on limited, readily available data from existing studies and limited, preliminary evaluation of the data. Figure 2 shows the primary product of this assessment, a preliminary hazard curve for peak horizontal acceleration (PHA) and relative contributions of the various seismic sources to the total PHA hazard. At this stage of the evaluation, existing data indicate that some faults in the immediate vicinity may need to be considered as potential earthquake sources and that the characterization of these faults strongly influences the results. Thus, issues such as surface faulting and secondary coseismic folding and faulting may also be of potential engineering significance to the proposed dam.

Results from the PSHA indicate that the Black Rock Project is located in an area of high seismicity and potential ground motions at the site are greatly influenced by the characterization of nearby seismic sources. Specifically, at return periods of about 10,000 years, total PHA is about 0.95 g. For motions greater than about 0.3 g, about 70% of the total hazard is derived from the current characterization of the Black Rock Valley fault. Cascadia seismic sources do not appear to be significant relative to the Black Rock Valley fault for the PHA hazard at the site for PHA of 0.3 or greater. However, these sources may be important at longer periods; periods which may be significant in more detailed analyses of engineered structures at the Black Rock site. Further evaluation of the potential seismic sources or identification of additional sources, may significantly alter the preliminary results developed in this study.

Reclamation typically designs its major power and pumping facilities for earthquakes having a return period of 2,500 years (2 percent probability of exceedance within a 50-year period), and assesses the risk of dam failure using an earthquake with a return period of 10,000 years. For the Priest Rapids Intake and Black Rock damsite, an earthquake having a return period of 2,500 years has a total PHA of about 0.50 g, and at a return period of 10,000 years, the total PHA is about 0.95 g.

Regionally, Black Rock damsite lies within the Yakima fold belt, a group of mostly east-west striking folds which formed during and subsequent to eruption of the Columbia River Basalts, about 10-15 million years ago. The geometry of the folds is consistent with activity shown by regional seismicity and stress data, which is dominated by north-south compression. However, there are significantly differing interpretations published in the technical literature regarding the origin and age of these folds that have profound implications for seismic hazard assessment. Despite these differing interpretations, one of the most critical issues in the hazard assessments is the proximity of the fault sources to the sites of interest. The relatively large PHA values contained in the present assessment are the direct result of the relative proximity of potential fault sources to the Black Rock site as compared to other sites that have had detailed seismic hazard evaluations in the region.

Initial geologic mapping indicates that a significant thrust fault is present in the right abutment of the proposed damsite. For the present characterization, this fault is included as part of the Black Rock Valley fault and considered as a potential earthquake source. If large earthquakes occur on this fault they could potentially be accompanied by up to several meters of surface faulting. The age and characteristics of this fault need further study for issues related to seismic source characterization at the site.

A hypothesis developed from currently in-progress mapping at the damsite indicates that the large fold on Horse Thief Mountain, the right abutment of the proposed dam, is related to the thrust fault that daylights in the lower portion of the right abutment and dips to the south beneath Horse Thief Mountain. Several secondary faults, scarps, and lineaments that appear to be related to secondary extension along the fold atop Horse Thief Mountain may be related to Quaternary deformation of this fault/fold. These features are also potential sites of coseismic secondary faulting, fissuring, and landslides.

Although not addressed in the initial probabilistic seismic hazard assessment, studies for potential reservoir-induced seismicity will be addressed in the future. The setting of site in a region of tectonic compression, very large and deep reservoir, and operations that may involve large fluctuations in depth and volume, all indicate that the probability of induced seismicity may be significant.

Hydrologic Hazard

A feasibility-level Probable Maximum Flood (PMF) Study was conducted by Reclamation to evaluate the hydrologic hazard associated with the potential Black Rock Reservoir. The results of this study are shown in Appendix C and summarized in Table 2.

Table 2 - Black Rock Dam - Feasibility Level PMF Study Summary

Flood Description	Peak Flow (cfs)	Volume (acre-feet)	Duration
Winter General PMP Storm with 100-yr antecedent rain flood (November to March)	20,200	29,100	10.5 days
Summer General PMP Storm with no antecedent flood (June to October)	28,900	28,700	3.5 days
Summer Local PMP Storm with no antecedent flood (June to October)	74,900	17,000	1-day

Reservoir Sizing Criteria

Based on the results of the Water Availability Study [5], two reservoir sizes were investigated for this assessment of project features. The Large Reservoir was sized for an active storage of 1,300,000 acre-feet and the Small Reservoir was sized for an active storage of 800,000 acre-feet. Aerial topographic data was used to develop elevation versus reservoir volume and area curves. See Figure 3. To reduce the dam heights required for total storage, the inactive storage was held to a minimum. To eliminate the need for a spillway, the PMF will be stored in the reservoir. Reservoir parameters for the large and small reservoirs are shown in Table 3.

Table 3 - Black Rock Reservoir Parameters

Design Parameter	Large Reservoir	Small Reservoir
Maximum Water Surface	Elevation 1778	Elevation 1712
Top of Active Water Surface	Elevation 1775	Elevation 1707
Active Capacity	1,300,000 af	800,000 af
Surface Area at Top of Active	13.5 sq. miles	10 sq. miles
Top of Inactive Water Surface	Elevation 1500	
Inactive Capacity	157,610 af	
Surface Area at Top of Inactive	3.25 sq. miles	

IV. Overview of Project Features

Large Reservoir Storage Options

Option 1: Large Reservoir - Pump Only Option

The Large Reservoir - Pump Only Option includes an intake with fish screens from Priest Rapids Reservoir, a 3,500 cfs pumping plant to lift the water to Black Rock Valley, a dam to store 1,300,000 acre-feet of active storage in Black Rock Reservoir, and a 2,500 cfs outflow tunnel and pipeline from the reservoir to Roza Canal. Table 4 summarizes the major features associated with this option between the Columbia River and Roza Canal. Figure 1 shows their relative locations as well as the locations of the distribution systems beyond the Roza Canal.

Option 2: Large Reservoir - Pump-Generating Option

In addition to the facilities identified for the Large Reservoir - Pump Only Option, the Large Reservoir - Pump-Generating Option includes a multi-level intake to selectively withdraw water from Black Rock Reservoir back to the Columbia River, a 3,500 cfs powerplant, and a 3,500 cfs tailrace channel to return the water back to Priest Rapids Reservoir. Table 5 summarizes the major features associated with this option between the Columbia River and Roza Canal. Figure 1 shows their relative locations as well as the locations of the distribution systems beyond the Roza Canal.

Small Reservoir Storage Option – Option 3

The Small Reservoir - Pump Only Option includes an intake with fish screens from Priest Rapids Reservoir, a 6,000 cfs pumping plant to lift the water to Black Rock Valley, a dam to store 800,000 acre-feet of active storage in Black Rock Reservoir, and a 2,500 cfs outflow tunnel and pipeline from the reservoir to Roza Canal. Table 6 summarizes the major features associated with this option between the Columbia River and Roza Canal. Figure 1 shows their relative locations as well as the locations of the distribution systems beyond the Roza Canal.

Table 4. Major Features of the Large Reservoir-Pump Only Option – Option 1

Priest Rapids Intake and Fish Screen
- Design Flow Capacity= 3,500 cfs
- Intake on right side of Priest Rapids Reservoir
- Normal Operating Water Surface Range= El. 481.5 to 488.0

Priest Rapids Pumping Plant
- Design Flow Capacity= 3,500 cfs
- Three 500 cfs, Two-stage spiral case pumps
- Two 1,000 cfs, Two-stage spiral case pumps

Inflow Conveyance System
- Design Flow Capacity= 3,500 cfs
- Two Conveyance Options: Discharge 1 - All Tunnel Option
  Discharge 2 - Tunnel/Pipe Option

Black Rock Dam
- Located on Original WIS Damsite
- Three Types: Concrete-face Rockfill, Central-core Rockfill, Roller-compacted Concrete

Black Rock Reservoir
- Active Storage: 1,300,000 af
- Inactive Storage: 157,610 af
- Top of Inactive Storage: El. 1500.0
- Top of Active Storage: El. 1775.0
- Maximum Reservoir Water Surface: El. 1778.0
- Spillway: None.
- Low Level Outlet: Dam type dependent.

Outflow Conveyance System
- Design Flow Capacity= 2,500 cfs
- Single level fish screened intake structure
- One Conveyance Option: Tunnel/Pipe Option

Black Rock Outlet Facility
- Located at MP 22.6 of Roza Canal
- One Bypass Structure Option: Design Flow Capacity= 2,500 cfs
- Two Powerplant Options: Option 1 - 1,500 cfs Design Flow Capacity
  Option 2 - 900 cfs Design Flow Capacity

Table 5. Major Features of the Large Reservoir - Pump-Generating Option – Option 2

Priest Rapids Intake and Fish Screen
- Design flow Capacity= 3,500 cfs
- Intake on right side of Priest Rapids Reservoir
- Normal Operating Water Surface Range= El. 481.5 to 488.0
- Separate tailrace channel to Priest Rapids Reservoir

Priest Rapids Pump/Generating Plant
- Pump Design Flow Capacity= 3,500 cfs
  - Three 500 cfs, Two-stage spiral case pumps
  - Two 1,000 cfs, Two-stage spiral case pumps
- Power Design Flow Capacity= 3,500 cfs
  - Two 1,750 cfs Francis turbines with 150 MW Generators

Inflow Conveyance System
- Design Flow Capacity= 3,500 cfs
- One Conveyance Option: Discharge 1 - All Tunnel Option
- Multi-level fish screened inlet/outlet structure

Black Rock Dam
- Located on Original WIS Damsite
- Three Types: Concrete-face Rockfill, Central-core Rockfill, Roller-compacted Concrete

Black Rock Reservoir
- Active Storage: 1,300,000 af
- Inactive Storage: 157,610 af
- Top of Inactive Storage: El. 1500.0
- Top of Active Storage: El. 1775.0
- Maximum Reservoir Water Surface: El. 1778.0
- Spillway: None.
- Low Level Outlet: Dam type dependent.

Outflow Conveyance System
- Design Flow Capacity= 2,500 cfs
- Single level fish screened intake structure
- One Conveyance Option: Tunnel/Pipe Option

Black Rock Outlet Facility
- Located at MP 22.6 of Roza Canal
- One Bypass Structure Option: Design Flow capacity= 2,500 cfs
- Two Powerplant Options: Option 1 - 1,500 cfs Design Flow Capacity
Option 2 - 900 cfs Design Flow Capacity

Table 6. Major Features of the Small Reservoir-Pump Only Option – Option 3

Priest Rapids Intake and Fish Screen
- Design Flow Capacity= 6,000 cfs
- Intake on right side of Priest Rapids Reservoir
- Normal Operating Water Surface Range= El. 481.5 to 488.0

Priest Rapids Pumping Plant
- Design Flow Capacity= 6,000 cfs
- Six 1,000 cfs, Two-stage spiral case pumps

Inflow Conveyance System
- Design Flow Capacity= 6,000 cfs
- One Conveyance Option: Discharge 1 - All Tunnel Option

Black Rock Dam
- Located on Original WIS Damsite
- Three Types: Concrete-face Rockfill, Central-core Rockfill, Roller-compacted Concrete

Black Rock Reservoir
- Active Storage: 800,000 af
- Inactive Storage: 157,610 af
- Top of Inactive Storage: El. 1500.0
- Top of Active Storage: El. 1707.0
- Maximum Reservoir Water Surface: El. 1712.0
- Spillway: None.
- Low Level Outlet: Dam type dependent.

Outflow Conveyance System
- Design Flow Capacity= 2,500 cfs
- Single level fish screened intake structure
- One Conveyance Option: Tunnel/Pipe Option

Black Rock Outlet Facility
- Located at MP 22.6 of Roza Canal
- One Bypass Structure Option: Design Flow capacity= 2,500 cfs
- Two Powerplant Options: Option 1 - 1,500 cfs Design Flow Capacity
Option 2 - 900 cfs Design Flow Capacity

The following sections describe each feature in detail.

V. Columbia River Intake

The Large Reservoir storage option requires a 3,500 cfs intake and pumping plant on the Columbia River to meet water delivery criteria, and the Small Reservoir storage option requires a 6,000 cfs intake and pumping plant on the Columbia River to meet water delivery criteria. In addition to these two options, a pump-generating option that permits Columbia River water stored in Black Rock Reservoir to be returned to the Columbia River to generate power was evaluated with the Large Reservoir storage option.

The potential site for the intake structure is located on the right bank of Priest Rapids Reservoir. See Figure 4. Priest Rapids Dam was constructed on the Columbia River between 1956 and 1961 and consists of left and right earth embankment sections, right bank gravity dam, two fish ladders, a gated spillway, and a powerhouse. The dam is operated by the Grant County Public Utility District and the active storage of the reservoir at maximum operating water elevation (488.0) is 48,600 acre-feet.

During the technical site review (Appendix A), the right bank of the Columbia River downstream from Priest Rapids Dam was evaluated for potential alternate intake locations. These downstream locations were limited to within four miles of the dam in order to avoid locating an intake within the environmentally-sensitive Hanford Reach of the Columbia River. Downstream of Priest Rapids Dam, the Columbia River is generally wide and shallow and the potential intake locations appeared to be less favorable compared to an inlet from the reservoir. Another consideration for placing the intake on the reservoir in lieu of the river downstream was to avoid the potential for significant daily fluctuation of water level. Discussions with Grant County Public Utility District personnel indicate that river fluctuations downstream of the dam can be as high as 14 feet, although operation of Priest Rapids Dam attempts to limit fluctuations to 5 to 6 feet. Daily fluctuation of Priest Rapids Reservoir is about 3 to 4 feet.

The intake for the Black Rock Project is located approximately 3,600 feet upstream from Priest Rapids Dam which provides adequate room for the physical layout of the intake, intake channel, pumping plant, switchyard, and tunnel portal, and also provides minimal impact to the existing embankment portion of Priest Rapids Dam. In addition to the stable water surface, locating the intake upstream of Priest Rapids Dam provides adequate hydraulic head for fish bypasses and adequate area for the fish screens, pumping plant, and switchyard. The upstream location of the intake will also minimize encroachment of the pumping plant facilities on the Wanapum Indian Village located downstream of the dam on the right side of the river.

Priest Rapids Intake and Fish Screen - 3,500 cfs Pump Only

Design Considerations

The design considerations for the intake channels and fish screens include hydraulic and biological criteria. The hydraulics must meet the maximum and minimum operating water surface elevations of Priest Rapids Reservoir and diversion flow requirements to Black Rock Reservoir. The maximum operating water surface elevation of Priest Rapids Reservoir is 488.0 feet, and the minimum operating water surface elevation is 481.5 feet. Intake facilities were provided with sufficient freeboard to prevent overtopping from the maximum water surface with flood surcharge, elevation 491.5 feet. Minimum intake pump submergence criteria for the pumping plant established the low point of the intake channel.

The National Marine Fisheries Services, Northwest Region criteria for Salmonids were used to design the fish screens and bypass pipes. These criteria include channel velocities, screen approach velocities, screen sweeping velocities, exposure time along screen, maximum bypass pipe flow velocity, and minimum radius of bypass pipe bends.

Structural loadings such as lateral earth pressure, uplift stability, seismic, and vehicle loads were not used at this level of design. These loadings would be a factor in the structural concrete thickness and reinforcing requirements for the walls. The structural concrete layouts and dimensions are based on past experience and designs of similar Reclamation fish screen structures.

Concept Description

From the intake at Priest Rapids Reservoir to the face of the pumping plant, the total length of the intake channel is approximately 2,366 feet. The channel consists of two different cross sections. See Figures 4 and 5. The initial 1,412 feet of the intake channel has three channel bays with vertical structural concrete walls. Two of the channel bays are sized for flows of 1,500 cfs each, and a third channel is sized for 500 cfs for a total of 3,500 cfs flow capacity. The channels were laid out with the top of concrete at elevation 495.50 feet and the invert elevation 468.00 feet. The channel depths are 27 feet 6 inches. The widths of the two 1,500 cfs channels are 36 feet 6 inches, and the 500 cfs channel is 15 feet wide. Currently the channel invert is assumed flat with no slope; however in final design, the channel inverts would have mild slopes to provide drainage when the channels are dewatered for maintenance. At minimum reservoir water surface elevation, the water depth in the channel is 13.5 feet and the maximum channel velocity is 4 feet per second (fps). At the maximum water surface, the flow velocities would be less than 4 fps.

Trashracks with an automated rake and a conveyor system are provided to collect trash at the inlet. This prevents large debris from flowing down to the fish screens and plugging them. A log boom may also be required in the reservoir in front of the trashracks but is not included in the current concept. Three top-sealed radial gates are provided at the reservoir intake to isolate the channels for emergency or short term maintenance of the fish screens and can also be used to regulate the downstream water surfaces. An access bridge deck is located over the inlet to allow access across the intake channel.

Bulkheads and guides are required at locations upstream and downstream of the structural intake channels. Each of the three structural channels will require sets of the bulkheads and guides in order to isolate each individual channel and still maintain the water diverting operation. Bulkheads will permit maintenance of the channels and associated mechanical equipment without shutting down the entire diversion. Mobile cranes were assumed to be available for installation and removal of bulkheads.

The fish screens are designed to meet the National Marine Fisheries Service (NMFS), Northwest Region, screen criteria for salmonid fry criteria. These criteria state that for salmonid fry, the approach velocity shall not exceed 0.40 fps. Approach velocity is defined as the water velocity component perpendicular and approximately three inches in front of the screen face. The total required submerged screen area (excluding area affected by structural components) was calculated by dividing the maximum diverted flow by the allowable approach velocity.

The fish screens for the Priest Rapids Intake are vertical flat panels installed within metal guide/support structures. The screen panels were assumed to be stainless steel wedge wire panels bolted to steel backing panels or supports. The NMFS screen criteria states that the screen slot openings (narrowest dimension) shall not exceed 0.0689 inches (1.75 mm). Adjustable baffles are provided in guides directly downstream of the screens to provide for uniform flow distribution over the screen surface. The fish screens will be cleaned by horizontal brush-type fish screen cleaners. Since the screens are designed for the maximum flow at the minimum operating water depth, metal barrier panels are provided above the screens to extend above the maximum design operating water surface.

To meet exposure time criteria, V-configurations of the fish screens were utilized for the 1,500 cfs option and fish screens in a single diagonal configuration were utilized for the 500 cfs channel. The following tables list the design criteria for the fish screens and the design values associated with the selected concept. Three 54-inch diameter bypass pipes are located at the end of the fish screens to deliver screened fish to the river channel below Priest Rapids Dam.

Table 7. 1,500 cfs Screen Design Parameters at Minimum Water Surface El. 481.5 feet

Fish Screen Parameters	**Screen Criteria Values**	**1,500 cfs Design Values**
Approach velocity	0.4 ft./sec.*	0.4 ft./sec.
Sweeping velocity	Greater than approach velocity	3.98 ft./sec.
Screen angle (from parallel with channel)	Less than 45^{o}	5.74^{o}
Exposure time along screen	60 to 90 seconds**	39 sec.
Screen length plus 10% for metal works	n/a	153 ft.

Table 8. 1,500 cfs Screen Design Parameters at Maximum Water Surface El. 488.0 feet

Fish Screen Parameters	**Screen Criteria Values**	**1,500 cfs Design Values**
Approach velocity	0.4 ft./sec.*	0.23 ft./sec.
Sweeping velocity	Greater than approach velocity	2.30 ft./sec.
Screen angle (from parallel with channel)	Less than 45^{o}	5.74^{o}
Exposure time along screen	60 to 90 seconds**	65 sec.
Screen length plus 10% for metal works	n/a	153 ft.

Table 9. 500 cfs Screen Design Parameters at Minimum Water Surface El. 481.5 feet

Fish Screen Parameters	**Screen Criteria Values**	**500 cfs Design Values**
Approach velocity	0.4 ft./sec.*	0.4 ft./sec.
Sweeping velocity	Greater than approach velocity	3.98 ft./sec.
Screen angle (from parallel with channel)	Less than 45^{o}	6.21^{o}
Exposure time along screen	60 to 90 seconds**	26 sec.
Screen length plus 10% for metal works	n/a	102 ft.

Table 10. 500 cfs Screen Design Parameters at Maximum Water Surface El. 488.0 feet

Fish Screen Parameters	**Screen Criteria Values**	**500 cfs Design Values**
Approach velocity	0.4 ft./sec.*	0.4 ft./sec.
Sweeping velocity	Greater than approach velocity	2.71 ft./sec.
Screen angle (from parallel with channel)	Less than 45°	6.21°
Exposure time along screen	60 to 90 seconds**	42 sec.
Screen length plus 10% for metal works	n/a	102 ft.

* Criteria for Salmonid fry.

**Not part of 1995 NMFS Criteria

Downstream from the fish screens, the three structural channels open to a single channel having a trapezoidal cross section. The bottom width of this channel is approximately 93 feet, side slopes are 1.5:1 (H:V), and the top of the channel is at elevation 500.0 feet. The channel would be lined with a 3.5 inch unreinforced concrete lining. The velocities in this section of the channel vary from 1.5 to 2.3 fps depending on the water surface elevation of the reservoir. This segment of the channel is approximately 608 feet long. The channel then widens and transitions to the pumping plant. The width of the channel at the pumping plant face is approximately 218 feet. The 346-foot long transition section is made at 10 degree angles parallel to the channel alignment and expands the bottom width from 93 feet to 218 feet. The transition invert is sloped vertically to meet the pumping plant intakes.

Maintenance roads are provided on each side of the channel. The access road on the left side (looking downstream) is 20 feet wide, and the access road on the right side is 12 feet wide. Guardrails would be provided for safety protection along the channel. Safety fencing would also be provided along both sides of the channel to protect against people and animals falling down the excavated side slopes.

Priest Rapids Intake and Fish Screens - 3,500 cfs Pump-Generating

This option has the same arrangement and criteria as described for the 3,500 cfs intake channel concept. The only change is the configuration of the intake area of the channel to accommodate a tailrace for the power generating units. See Figures 6, 7, and 8.

The tailrace channel for the power generating side of the plant is located between the intake channel and Priest Rapids Reservoir and is approximately 483 feet long. The physical layout of the channel consists of a transition from the powerplant face to a 35-foot wide structural channel with vertical walls. The top of the concrete walls are at elevation 500.5 feet, and the invert elevation is at 471.50 feet.

The design velocity of the channel is 10 fps during the minimum water surface elevation with the maximum discharge of 3,500 cfs. The velocity decreases as the reservoir water surface rises to the maximum water surface. The channel layout selects the shortest possible path back to the reservoir. This short layout was selected for its minimal channel friction loss however, the short tailrace channel will pass through the embankment section of Priest Rapids Dam. Cutoffs are provided to prevent seepage between the embankment and structure.

Two 20-foot wide maintenance roads are provided on either side of the tailrace channel. The channel has an access bridge aligned with the centerline of the existing Priest Rapids Dam embankment. A deck is also provided at the location of the tailrace bend downstream of the channel transition.

Priest Rapids Intake and Fish Screen - 6,000 cfs Pump Only

This concept is similar to the 3,500 cfs concept except that it consists of four channel bays of 1,500 cfs. See Figures 9 and 10. Each channel bay is approximately 36 feet 6 inches wide. The total length of the channel is approximately 2,340 feet which includes 1,433 feet of structural concrete channels, 727 feet of excavated, unreinforced, concrete-lined, trapezoidal cross section channel, and 180 feet of transition to the pumping plant.

The excavated trapezoidal channel for this concept has a bottom width of 154 feet and side slopes of 1.5:1. The section then transitions to the pumping plant both horizontally and vertically to accommodate the plant width and pump submergence requirements.

All required appurtenances are similar to the 1,500 cfs channel bays of the 3,500 cfs concept. There are four sets of bulkheads and guides for both upstream and downstream use. There are also four top sealed radial gates. The access bridge and maintenance roads are the same widths. Since there are four bays, the total channel width is greater than the 3,500 cfs concept. Trashracks and rakes are increased to accommodate the extra bay for this concept. Guardrails and safety fencing will be required and will be similar to the 3,500 cfs concept.

Fish screen criteria and design values meet those shown in Tables 7 and 8 for the minimum and maximum water surface elevations. Lengths and locations of the bypass pipes and outfalls are assumed the same as the 3,500 cfs concept, except that there will be four 54-inch diameter bypass pipes for the four fish screen bays.

Priest Rapids Pumping Plant and Switchyard - 3,500 cfs

Design Considerations

The location of the pumping plant and service yard was controlled by the intake channel location, fish bypass requirements, location and alignment of the tunnel portal to the discharge line, space requirements for the plant and switchyard, access into and around the plant, and access into the Service Bay. See Figure 11. The service yard was set at elevation 507.5 feet for compatibility with the existing ground elevation and to reduce the visibility of the plant structure and switchyard from the Wanapum Indian Village. Reclamation decided to go with a lower structure for the Unit Bay and raised superstructure for the Service Bay to keep the service yard at an elevation more compatible with the existing ground surface and to allow access and handling of equipment into and out of the building structure from the service yard elevation. The lower profile for most of the superstructure also helps mitigate the structural demand for lateral earthquake loads. These loads are anticipated to be significant based on available seismic data.

Pumping units with capacities of 500 cfs and 1,000 cfs were selected to accommodate months when downstream flow targets in the Columbia River would restrict the volume of water that could be pumped from the river. Besides flexibility of operations, more smaller units reduces the submergence requirements for the units and permits unit maintenance without sacrificing a large percentage of the plant capacity. The lift from Priest Rapids Reservoir to Black Rock Reservoir is very high (approximately 1400 feet steady state) and the size of the units led to the use of spiral case pumps. Pumping could be accomplished by single-stage or two-stage pumps, however because of the high pumping head, submergence requirements for a single-stage unit are much greater than requirements for a two-stage unit. For single-stage pumps the required submergence is on the order of 180 feet, while for two-stage pumps the submergence requirement is approximately 60 feet. Two-stage pumps were selected for this study to reduce the depth of excavation for the pumping plant.

Concept Description

The pumping plant is a reinforced concrete structure approximately 375 feet long by 163 feet wide. The indoor type structure will house five units: three 500 cfs units with 98,000 hp motors, and two 1000 cfs units with 200,000 hp motors. The rated head of the pumps is 1,400 feet. The pumping units require 62 feet of submergence below the minimum intake water surface elevation of 481.5 feet which set the centerline elevation for the lower stage of the pump impeller at elevation 419.5 feet. See Table 11 for unit data. Handling requirements for the rotor/shaft assembly controlled building and overhead crane elevations and the estimated weight of the rotor/shaft assembly (700,000 lbs) controlled the selection of two 200-ton overhead cranes acting in tandem in the Unit Bays. In the Service Bay, a 100-ton overhead crane is provided. Space was provided in the plant for unit disassembly, auxiliary mechanical, and electrical equipment. Precast concrete double tees were selected for the roof structure based on span and anticipated availability. See Figures 12 through 17 for 3,500 cfs pumping plant general arrangement details.

Table 11. Priest Rapids 3,500 cfs Pumping Plant Unit Data

Unit Data	**500 cfs Unit**	**1, 000 cfs Unit**
Number/Type of Units:	3 - Two-stage spiral case	2 - Two-stage spiral case
Design Discharge:	500 cfs	1,000 cfs
Design Head:	1,400 feet	1,400 feet
Min. Impeller Submergence	62 feet	62 feet
Max. Spiral Case Dimension	18.2 feet	26.0 feet
Top of Suction Tube Invert	El. 468.0	El. 468.0
Guard Valve:	60-inch spherical	78-inch spherical

Due to the high head at the plant, the pump discharge valves were assumed to be heavy spherical valves supplied by the pump manufacturer.

Steel piping for the tunnels, manifolds, and penstocks were designed in accordance with AWWA M11 [9] and ASCE Manuals and Reports on Engineering Practice No. 79 [10]. The large diameter tunnel liners and high pressure manifolds were designed with ASTM A572 Grade 50 steel plate. All other manifolds, penstocks, and outlet works piping were fabricated from ASTM A36 steel plate. The discharge line earthwork quantities were calculated based on the typical pipe trench section shown on Figure 32.

The estimated size and weights of the synchronous machines (generators and motors) were based on existing Reclamation machines having similar speed and kVA or horsepower ratings. Standard 3-phase utilization voltages of 480; 6,900; and 13,800 volts were assumed for the study. Because of the high continuous current loads at the Priest Rapids Pumping Plant, high capacity isolated-phase bus was used in the estimate. All the 15 kV switchgear utilized SF6 type unit circuit breakers.

The estimate also assumes that a new 500-kilovolt line will be constructed from the Midway Substation located 6 miles east of the Priest Rapids Pumping Plant and Switchyard. The connection at the Midway Substation includes a circuit breaker and disconnect switches. The pumping plant switchyard will include transformers, circuit breakers, and disconnect switches.

Priest Rapids Pump-Generating Plant and Switchyard - 3,500 cfs

This option has a similar arrangement as described in the 3,500 cfs Pump Only option except, the structure and yard have been expanded to accommodate two turbines and a tailrace channel adjacent to the pumping plant intake channel. The offset height of the superstructure between the Unit Bay and Service Bay and the use of two separate crane levels has also been eliminated from this option because the turbine and generator elevation settings do not facilitate taking advantage of this arrangement. A single crane and roof level are shown for this option. See Figures 18 through 25.

One of the unfortunate results of choosing two-stage pumps to deliver the water is that it made the use of pump-turbines less attractive. Similar size two-stage pump-turbines have been built but they are near state-of-the-art and maintenance requirements are expected to be high. For these reasons, separate turbines were selected for the pump-generating option.

The pump-generating plant is a reinforced concrete structure approximately 490 feet long by 163 feet wide. The structure will house five pumping units: three 500 cfs units with 98,000 hp motors, and two 1000 cfs units with 200,000 hp motors, and two Francis turbine units each with a design discharge of 1,750 cfs. Pump unit data is shown in Table 11 and turbine data is shown in Table 12. Handling requirements for the pumping units controlled over the handling requirements for the turbines.

Table 12. Priest Rapids 3,500 cfs Powerplant Unit Data

Unit Data	1,750 cfs Unit
Number/Type of Units:	2 Francis
Design Discharge:	1,750 cfs
Design Head:	1,130 feet
Speed:	400 rpm
Assumed Unit Efficiency	90 percent
Power Output at Design Head:	150 MW
Min. Turbine Submergence	29.8 feet
Max. Scroll Case Dimension	26.5 feet
Bottom of Draft Tube	El. 431.5
Guard Valve:	102-inch spherical

Priest Rapids Pumping Plant and Switchyard - 6,000 cfs

This option has a similar arrangement as described for the 3,500 cfs Pump Only option except the structure has been expanded to accommodate six 1,000 cfs pumping units. See Figures 26 through 31.

Construction Considerations

Construction Access: The current access to the right side of Priest Rapids Reservoir is across Priest Rapids Dam. The clear width and sharp turns across the spillway deck would restrict movement of large construction equipment across the dam and an alternate means for construction access, and future operation and maintenance access was developed along the right side of the Columbia River from State Highway 24 (SH24) to the Intake facilities. The proposed road follows the alignment of the abandoned railroad tracks.

Cofferdam: A cofferdam will be required in the reservoir to permit construction of the intake. Our estimate assumed a circular-type, cellular cofferdam would be constructed. A cellular cofferdam is a gravity retaining structure formed by a series of interconnected straight web steel sheet pile cells filled with free draining granular soil. The circular-type cofferdam consists of individual large diameter circles connected together by arcs of smaller diameter. The

380-foot long cofferdam was assumed to be constructed with 32-foot-diameter cells that are 28 feet high. The top of the cofferdam was assumed to be at elevation 488.0 feet.

Excavations: The depth of overburden over sound basalt was assumed to be 20 feet. All rock excavation was assumed to have a 0.25:1 cut slope and overburden earth excavation is assumed to have 1.5:1 cut slope. Based on available geologic and groundwater data, dewatering is not necessary because groundwater is not likely to seep into the foundation excavation.

VI. Inflow Conveyance System

Inflow Conveyance System - 3,500 cfs Pump Only

Two different inflow conveyance systems were analyzed to transport the flow of 3,500 cfs from Priest Rapids Reservoir to Black Rock Reservoir. Both alignments encroach on the Yakima Firing Center Military Reservation to some extent. See Figures 1 and 32. The first option, Discharge 1, is an all tunnel option with a 16-foot-diameter manifold connecting to a 17-foot-diameter tunnel sloping steadily up towards the reservoir. The tunnel has a 22-foot-diameter surge shaft located 3,850 feet from the pumping plant and extending to El. 2106. See Figure 33. The second option, Discharge 2, is a tunnel/pipe option comprised of a 16-foot-diameter discharge pipe and tunnel, 16-foot-diameter vertical shaft, 21-foot-diameter gravity tunnel, and an 18-foot-diameter pipe transitioning to a 17-foot-diameter pipe connected to the low level outlet works at Black Rock Dam. See Figure 34.

General Design Considerations

The tunnel and pipeline diameters were sized using a flow of 3,500 cfs. The transient design was based on an additional 5% (3,675 cfs) to account for the pump wear factor and manufacturer's allowance. The Priest Rapids Reservoir water surface elevation used for the hydraulics and transient study was El. 488 (Normal Maximum). A Black Rock Reservoir level of El. 1500 (Top of Inactive) was used to calculate the maximum downsurge; to size the tunnel and pipeline diameters; and to size the surge tank diameter. A Black Rock Reservoir level of El. 1775 (Top of Active) was used to calculate the maximum upsurge pressure at the pumping plant and to determine the elevation required at the top of the surge tank.

The following factors and assumptions were used for the hydraulic and transient analysis of the 3,500 cfs inflow conveyance systems:

Design Flow:	3,500 cfs
Transient Design Flow:	3,675 cfs
Colebrook White Rugosity Value:	0.002 (Tunnel)
Colebrook White Rugosity Value:	0.001 (Pipeline)
Downsurge pressures:	No negative pressures.
Rated head:	1,450 feet
Speed:	400 rpm
WR^2:	5,000,000 per unit
Efficiency:	0.85
Pumps:	5 equal two-stage pumps*

* Five equal units were assumed to simplify the transient analysis.

Discharge 1 (All Tunnel) - Hydraulic Design Considerations

Initially, the tunnel was designed with a shaft similar to the WIS Report [2], however, the design was later simplified to have a tunnel with a constant slope from Priest Rapids to Black Rock Reservoir. The tunnel portal was located just outside of the pumping plant switchyard fence with the centerline at El. 495. Based on the transient analyses, the end of the tunnel was located at El. 1440 to prevent a negative downsurge in the tunnel near the Black Rock Reservoir. See Figure 33.

The top of the surge shaft was set at approximate El. 2106 to prevent overtopping and was located to provide a level spot for construction activities. The top elevation is based on upsurge from the pumping plant shutdown while Black Rock Reservoir is at El. 1775. The maximum design grade line at the pumping plant was determined to be El. 2800 which converts to 2,370 feet of pressure based on a pump/pipeline centerline elevation of 430 feet. The steady state hydraulic grade line at the pumping plant, not including friction loss through the pumps and manifold, was elevation 1876.0 feet. See Figure 35 for a graphical representation of the design and hydraulic grade lines for Discharge 1.

Discharge 2 (Tunnel/Pipe) - Hydraulic Design Considerations

The design of the tunnel/pipe initially was to be a pressurized system from the pumping plant to Black Rock Reservoir. However, after review of the hydraulic and transient analyses, increasing the size of the tunnel after the surge shaft and using a gravity tunnel provided a more efficient and effective solution. The downstream tunnel diameter was increased to 21 feet which provided a normal depth of 15.1 feet and slope of 0.0015 using a Manning’s n of .0015. The elevation of the gravity tunnel downstream portal was based on the hydraulic grade line required to allow the 3,500 cfs to flow around Yakima Ridge in an 18 foot-diameter pipe at approximately El. 1800. Beyond the south side of the Yakima Ridge, the pipe diameter was reduced to 17 feet. See Figure 34.

The manifold and initial tunnel diameter were sized to minimize the diameter while maintaining a flow velocity below 20 fps. A 16-foot-diameter discharge pipe and tunnel have a flow velocity of approximately 18 fps at 3,500 cfs. The maximum design grade line at the pumping plant was determined to be El. 2700 which converts to 2,270 feet of pressure based on a pump/pipeline centerline elevation of 430 feet. The steady state hydraulic grade line at the pumping plant, not including friction loss through the pumps and manifold, was elevation 1910.0 feet. See Figure 36 for a graphical representation of the design and hydraulic grade lines for Discharge 2.

Tunnel Design Considerations

Appraisal designs for the tunnel support were based on existing geologic design data and Chapter 4 of Reclamation's Design Standards No. 3 [11]. The tunnel designs include initial and final support. The initial support holds the tunnel opening stable until installation of the final support, or lining. The contractor will install the initial support immediately after advancing the heading. Figures 37 and 38 show the initial and final support used in this Assessment.

Tunnel excavation will probably be accomplished using Tunnel Boring Machines (TBMs). Basalt generally does not preclude this method of excavation and the design does not anticipate unusual bit (disk cutter) wear. Shorter tunnels with lengths less than 4,000 feet may be excavated by drill and blast methods. Intermediate length tunnels may be excavated by either method, depending on the particular contractor's resources.

Most shaft excavation will probably be by full raise bore, raise bore and slash down, or raise bore and ream down excavation methods. The shaft excavation on the outlet tunnel will be by raise bore and slash down as the diameter precludes using shaft boring machines or raise boring to the final diameter. Raise bore excavation is accomplished by boring a small hole from the surface, removing the small boring cutter head, and attaching a boring head with the same diameter as the final excavated diameter of the shaft to the drill steel at the shaft bottom, then pulling the boring head upward to the surface. The operation removes the drilling waste from the small hole at the shaft top and miners remove shaft waste from the raise bore and shaft bottom. The per unit cost of removing waste at the shaft top is much higher than at the shaft bottom.

The raise bore and slash down method begins similar to the full raise bore except the raise bore is a smaller diameter than the shaft's final excavated diameter. Miners then remove the boring head and drill steel at the shaft top and begin slashing down using drill and blast, or other techniques to excavate to the final shaft diameter. The shaft waste falls through the raised bored hole and down to the tunnel below, where miners remove it. The raise bore and ream down method is similar to the raise bore and slash down method except the final shaft diameter is excavated by reaming the raised bored hole to a larger diameter.

Water is always a major concern in tunneling, however all of the potential tunnels are above the current water table so groundwater should not be a major problem. Surface waters coming from rains will eventually enter the tunnel. All tunnels can be excavated uphill, alleviating minor water problems. The initial tunnel support will depend on the intercepted geology, and may be interdependent with the final lining for a particular reach. For this Assessment, rock quality was assumed in order to determine what initial support would be required and general lengths of reaches of structural steel tunnel supports along with a percentage of the remaining tunnel that needed patterned rock reinforcement, spot rock reinforcement, or no reinforcement were determined.

Final tunnel lining design was based on Reclamation design standards. While the mineralogy of the rock indicates that an unlined tunnel is possible, the design uses a lining to assure reasonable hydraulic friction and account for areas where the rock may be highly fractured. While the tunnel design calls for mostly unreinforced lining, the design prescribes some reinforcement to curtail leakage in highly fractured reaches in the pressure tunnels. Tunnel design also incorporates steel lining at the portals to insure water tightness as the tunnel nears the surface. This steel liner will be backfilled with concrete and grouted. The tunnels will not require steel lining when the portal is under Black Rock Reservoir.

Pipeline Design Considerations

The pipeline for the Tunnel/Pipe option (Figure 34) was designed using steel pipe and AWWA M11 guidelines [9] for internal pressure. The steel pipe wall thickness was sized using a design pressure based on the water surface at the end of the tunnel, El. 1881.0. The design pressure was not increased for transient pressure because the valves in the dam's low level outlet works at the end of the pipeline would be able to close slow enough to prevent harmful pressure surges. An allowable design stress of 18,000 psi was used which is 50% of the minimum yield point of ASTM A36 steel. For this study, the pipeline was divided into three sections and designed for the parameters shown in Table 13. Future studies would further refine the steel pipe design, hydraulics, and transient pressures.

Table 13. Discharge 2 Pipeline Design Parameters

Stations	Diameter Feet (inches)	Design Pressure psi	Pipe Wall Thickness, inches
154+50 to 415+60	18 (216)	168	1.0
415+60 to 465+60	17 (204)	165	1.0
465+60 to 545+00	17 (204)	229	1.3

The pipeline vertical alignment was based on a minimum cover of 5 feet. The average cover depth was 13 feet. During feasibility design, the pipeline vertical profile could be refined and the average cover depth decreased. The side slopes and bench widths were determined based on a review of available geologic data. See Typical Pipe Trench Section in Figure 32.

At Black Rock Reservoir, the pipeline was assumed to connect to the low level outlet works pipe so that both the low level outlet works and pipeline would use the same tunnel through the dam abutment.

Reservoir Outlet Design Considerations

Given the anticipated high flows (3,500 to 6,000 cfs) and velocities during the initial filling of the Black Rock Reservoir inactive pool space, provisions for erosion control/protection of the reservoir rim were included in the estimates. The inflow conveyance systems enter the reservoir approximately 100 feet above the valley floor at a distance of approximately 4,000 feet from the valley bottom. Reclamation decided that some type of conveyance would be needed to minimize erosion of the hillside until the reservoir reaches the inlet elevation. For this Assessment, a short open channel was assumed to direct flow from the outlet structure into a large diameter (20- to 24-foot) steel pipe. The pipe would carry the flow downhill towards the valley bottom where it would terminate with a 90-degree upward bend. A large concrete thrust block would be necessary to handle the thrust loads at the end of the 90-degree bend. No attempt to optimize alternatives or costs for this erosion control plan was done at this level of study, however this should be completed as part of any future studies.

Inflow Conveyance System - 3,500 cfs Pump-Generating

For this study, the inflow conveyance system for the Pump-Generating option was assumed to be the same as the All Tunnel conveyance system for the 3,500 cfs pump only option. (Discharge 1). One difference is the need for a multi-level intake structure at Black Rock Reservoir to control the withdrawal elevation for water returning to the Columbia River to meet as yet to be determined water quality objectives. For this Assessment, the proposed Multi-Level Inlet/Outlet Structure is a free-standing tower with fixed intakes at elevations 1450.0, 1540.0, 1630.0, and 1720.0 feet. These ports are valve controlled so any combination of withdrawal levels can be achieved. The intakes discharge into a wet well before entering the 17-foot-diameter tunnel to Priest Rapids Pump-Generating Plant. See Figures 39 and 40.

Fish screens would be installed at each intake level. Fish screen sizing criteria was assumed to be the same criteria used to size the intake structure at Priest Rapids Reservoir. The fish screens would be stationary, half-cylinder-shaped screens with flat side panels attached to the intake tower. The fish screens will only be used when in the generating mode. However, cleaning of these screens was assumed to be by passing pumped (back flush) water through the screens for a short period when in the pumping mode. During normal pumping operations, pumped water will be discharged through two 17-foot by 17-foot gates located at the bottom of the wet well.

Inflow Conveyance System - 6,000 cfs Pump Only

Only one conveyance system was analyzed to transport the 6,000 cfs flow from Priest Rapids Dam to Black Rock Reservoir. A 22-foot-diameter tunnel sloping steadily up towards the reservoir was used. The tunnel has a 22-foot-diameter surge shaft located 4,050 feet from the upstream end of the tunnel, extending to El. 2107. See Figure 41.

General Design Considerations

The tunnel diameter was sized using a design flow of 6,000 cfs. The transient design was based on an additional 5% (6,300 cfs) to account for the pump wear factor and manufacturer's allowance. The Priest Rapids Reservoir water surface elevation used for the hydraulics and transient study was El. 488 (Normal Maximum). A Black Rock Reservoir level of El. 1500 (Top of Dead Pool) was used to calculate the maximum downsurge; to size the tunnel diameter; and to size the surge shaft diameter. A Black Rock Reservoir level of El. 1775 (Top of Active) was used to calculate the maximum upsurge pressure at the pumping plant and to determine the minimum elevation at the top of the surge shaft.

The following factors were used for the hydraulic and transient analysis for the 6,000 cfs inflow conveyance system:

Pipeline Design Flow:	6,000 cfs
Transient Design Flow:	6,300 cfs
Colebrook White Rugosity:	0.002 (Friction Loss Coefficient)
Downsurge pressures:	No negative pressures.
Rated head:	1450 feet
Speed:	400 rpm
WR^2:	5,000,000 per unit
Efficiency:	0.85
Pumps:	6 equal-sized two-stage pumps

Tunnel design considerations are similar to those discussed for the 3,500 cfs Pump only Option.

Discharge 1 (All Tunnel) - Hydraulic Design Considerations

The tunnel was designed similar to the 3,500 cfs tunnel design which has a constant slope from Priest Rapids to Black Rock Reservoir. The tunnel portal is located just outside of the pumping plant switchyard with a centerline at elevation 495. Based on the transient analyses, the end of the tunnel was located at elevation 1440 to prevent a negative downsurge in the tunnel near the Black Rock Reservoir. The top of the surge shaft was set at approximate elevation 2107.0 to prevent overtopping and was located to provide a level spot for construction activities. The maximum design grade line at the pumping plant was determined to be El. 2650 which converts to 2,220 feet of pressure based on a pump/pipeline centerline elevation of 430 feet. The steady state hydraulic grade line at the pumping plant, not including friction loss through the pumps and manifold, was elevation 1782.0 feet. See Figure 42 for a graphical representation of the design and hydraulic grade lines for Discharge 1.

Construction Considerations

Pipe Fabrication: Steel pipe sections 16-feet in diameter and greater will have to be transported in pieces by truck or rail and welded together and pressure tested in the field.

Tunnel Excavation: The potential for varying rock quality encountered during tunnel excavation will necessitate a flexible working relationship with the contractor. Differing ground, water and gas conditions from assumed conditions will affect the tunnel construction. Based on current knowledge of the tunnel alignment, bedrock will consist of a series of basaltic lava flows and associated interflow sediments. The basaltic portion of the tunnel will be in rock which will likely vary structurally and texturally from massive nonvesicular to highly vesicular and flow-breccia types. Soft, relatively uncemented interflow sediments will also likely be encountered. A number of shear zones, consisting of fractured rock with soft gouge materials may also be encountered during tunneling.

VII. Black Rock Dam and Reservoir

Large Embankment Dams

Design Considerations

There are a number of design considerations associated with the construction of a large embankment dam in the Black Rock Valley. None of these considerations are viewed to be “fatal flaws” that would indicate the site is not technically feasible. Rather, it is believed that a safe dam could be constructed, and that no unusual measures or features beyond what is typically done for a major embankment dam would be required. Nonetheless, there are a number of issues that will need special attention during design, and some of the most significant are listed below.

High Seismicity

Black Rock Dam would be located in an area of high seismicity, or earthquake potential. The presence of the Black Rock Valley fault is the largest contributor to the seismic hazard, although there are a large number of other contributing earthquake sources. Two additional notable ones are the Yakima Ridge East fault, the second largest contributor to the hazard after the Black Rock Valley fault, and the deep zone of the Cascadia Subduction Zone, capable of producing very large magnitude earthquakes. Based on Reclamation's initial probabilistic seismic hazard assessment (Appendix B), the peak horizontal ground acceleration expected from a 10,000-year earthquake has a mean value of 0.95g. This is a large ground motion and dictates that a dam needs to be able to resist significant earthquake shaking.

This high level of shaking leads to the potential of causing lower density embankment or foundation saturated soils to experience liquefaction, which is essentially a loss of strength that can result in large slope failures. To mitigate this concern, it is critical that all potentially liquefiable foundation soils are removed and that all embankment materials are compacted to high densities, which can be routinely accomplished through the use of large rollers.

Another potential concern is that earthquake shaking, if severe and long enough, can induce slope failures in an embankment. This concern can be addressed by carefully analyzing the dam for potential deformations from the expected earthquake load, and designing crest dimensions, zoning, and embankment slopes to ensure stability, as well as selecting strong materials and keeping the phreatic surface (water level) in the embankment as low as possible.

Potential Fault Displacements

Preliminary investigations indicate that at least one significant thrust fault is located within the proposed footprint of the dam, at the base of the right (south) abutment. This fault has not been studied in sufficient detail to define its activity or the magnitude of potential displacements. At this stage of study and based on available information, it can only be assumed that fault offsets within the dam footprint are possible, and that such displacements could range from a few centimeters to several meters. Given the orientation of the east-west folds comprising the Yakima Fold Belt which includes Black Rock Valley, the orientation of the displacements would be in the north-south (cross valley) direction, reflecting compression of the folds. From a dam engineering standpoint, this orientation would likely be favorable over an upstream-downstream displacement which would create transverse cracks through the dam. However, severe cracking would still likely result from a significant fault offset.

The potential for and impact of such potential displacement of a fault at the right abutment must be considered and accommodated in the dam design. Because an embankment dam is generally viewed as less stiff or rigid than a concrete dam, an embankment alternative may be best able to accommodate potential fault displacements. Key features to include in an embankment would be filters and drains of sufficient dimension to ensure that cracking, offsets, or differential movements will not exceed the width of the filters. These filters and drains should be constructed of clean, cohesionless, and permeable sands and gravels so that if the dam is cracked, these materials will collapse or rearrange so that a crack is not supported within these zones. While the upstream water barrier (an earth core or concrete face, for example) would be expected to crack and possibly stay open from a fault offset, the filter would serve to ensure that no fine-grained materials from a core would be able to erode downstream. The gravel drain located downstream from the filter would provide for safe collection of any seepage that is passed through the crack in the earth core or concrete face. In addition, filters placed upstream of the water barrier may serve as crack "pluggers" that introduce sand into cracks in the water barrier to help seal the cracks.

Another design feature frequently utilized when fault displacement is possible is the use of large rockfill shells. These rockfill shells, constructed of rock up to 3-foot size, form an extremely stable downstream buttress for the earth core or concrete face. Of equal importance is the proven ability of rockfill to allow extensive reservoir leakage or flows to safely "flow through" the rockfill without causing dam failure. This is possible because of the high horizontal permeability of rockfill and the fact that extremely high seepage velocities are required to erode or move large size rocks.

Varying Rock Quality

The bedrock forming the abutments and valley section beneath Black Rock Dam consist of interbedded and folded volcanic and sedimentary rocks of the Columbia River Basalt Group. In essence, these are a series of basalt flows that were extruded and flowed over the Columbia Basin between 18 and 6 million years ago. Individual flows were up to 100 feet thick, and the time periods between sequential flows was from hundreds to tens of thousands of years, which allowed for sediment deposition between basalt flows. As a result, the bedrock stratigraphy consists of a number of different basalt flows with sedimentary interbeds separating some of these flows. In addition, due to the nature of the flow deposition, the basalts may contain sediments that are "rafted" within the basalt or contain "pillow" structures that also contain pods of fine sediment and fractured basalt. It is not unusual to see "interflow zones" of higher permeability at the top or bottom of flows due to shearing and intermixing during deposition or resulting from differences in cooling of the flows.

As the bedrock surface is excavated during construction, it would be expected that rock quality could vary significantly as different areas of one flow or different flows are uncovered. This is by no means a significant detriment for an embankment foundation, but does mean some flexibility will be needed during construction to ensure a suitable foundation is reached. Considerable onsite geologic and geotechnical presence will thus be needed to determine the adequacy of the bedrock and the degree of foundation treatment measures such as additional excavation, slush grouting, and filter placement.

In addition, the varying bedrock composition and quality will require additional investigations during advanced design phases to better understand the bedrock properties and permeability (fracture density, openness, infilling characteristics, etc.), to develop a foundation grouting program, to explore foundation conditions, and to potentially reduce bedrock seepage.

Thick Overburden Deposit of Varying Composition

Geologic explorations by both WIS and Reclamation have confirmed the presence of a thick deposit of overburden overlying the basalt bedrock. Drill holes have indicated that the overburden in the Black Rock Valley is deeper than 200 feet near the base of the right abutment, and may average more than 100 feet deep across the right center portion of the valley. Although the overburden includes some surficial loess (wind-blown silt), colluvium, and recent alluvium, the large majority appears to consist of the Ringold Formation. Based on the limited explorations, the Ringold appears to be a highly variable deposit, consisting mainly of basalt gravels and cobbles in a sand with fines matrix, but also including significant layers of fluvial sand, silt, and clay. These varying materials have been described as poorly to well-consolidated, and poorly- to semi-indurated.

An obvious design consideration with this type of overburden is determining how much to remove beneath the dam. On one hand, a reasonable but perhaps conservative approach would be to remove all of the overburden down to bedrock beneath the entire footprint of the dam. Given the significant height of the proposed Black Rock Dam, the relatively steep slopes of the embankment, and the high seismicity of the site, this option would certainly be defendable. On the other hand, portions of the Ringold, particularly the deeper layers, are described in the drill logs as a dense, indurated deposit of gravels and cobbles. This type of material would be expected to be a firm and non-liquefiable foundation capable of supporting a large dam, although some type of cutoff to bedrock may be needed to minimize and protect against seepage. During advanced design phases, additional characterization of the Ringold Formation will be important in helping to determine the optimum definition (a blend of technical and economic considerations) of the amount of Ringold to remove beneath Black Rock Dam.

<u>Construction Material Availability</u>

A key consideration for the design of any embankment dam is utilization of available materials. The nature and availability of construction materials is important for both technical and economic reasons. For a dam the size of the proposed Black Rock Dam, it will be important to secure high quality materials for the key zones in the embankment. If such materials are located reasonably nearby, that is a large economic advantage since the costs of hauling large volumes of materials can be huge. In addition, since potentially large volumes of materials will be generated from excavation of much, if not all, of the foundation overburden, an ideal embankment design would include the use of those materials in a non-critical zone as opposed to wasting them.

<u>Large Dam Height</u>

Inherent in some of the above considerations, but worth emphasizing separately, is that the proposed Black Rock Dam will be a very high embankment, approaching 800 feet in structural height. Although well within the precedents set in terms of high embankments, this dam will nonetheless deserve special attention due to its height and the large hydraulic head behind it. Large site investigation and materials testing programs will be needed to ensure the site conditions are well understood. Detailed analyses will be critical to ensure a safe design is developed. In addition to these measures, such a design would need to be independently reviewed by an expert board of consultants.

<u>Type of Embankment Dam</u>

Given the design considerations listed above, an initial step in the design process is to select the appropriate type or types of embankment dam to consider for this damsite. A rockfill dam is an obvious choice for the Black Rock site, and better suited than a zoned earthfill embankment for several reasons. First of all, there is a relative lack of impervious soils or even unconsolidated pervious soils at the damsite. The only immediately available impervious soil is the relatively thin loessial layer that blankets much of the site, particularly the higher areas. Much of the foundation overburden consists of the Ringold Formation, which is several million years old, somewhat variable in material composition (clays, siltstones, and conglomerates to name a few), and often cemented. Extensive development of the Ringold as a borrow source would likely lead to a wide range of material properties and differing degrees of difficulty of excavation. Basalt, however, is present throughout the dam and reservoir area, with relatively little soil cover on the abutment and reservoir rims. The basalt, through quarrying, provides an unlimited source of rockfill.

Secondly, the proposed damsite is in an area of relatively high seismicity. Based on the PSHA (Appendix B), the expected 10,000-year ground motion at the site is 0.95g. Furthermore, there is some potential that displacements could occur on faults that pass beneath the dam alignment. Such high ground motions and the potential for fault movement require a dam type that is seismically stable even under very large loadings. Rockfill dams are recognized to be one of the best dams under these conditions, primarily because their design affords a large downstream portion that remains unsaturated and strong, and yet provides permeability to let seepage pass through in the event that the impervious element of the dam is cracked or similarly damaged. Later paragraphs will describe two different rockfill embankments that appear best suited to the Black Rock site – a concrete face rockfill dam and a central core rockfill dam.

<u>Axis Location</u>

Two potential alignments were explored during this assessment study. The original, or farthest east (downstream) alignment, was initially proposed and explored by WIS, while an alternate alignment located further west (upstream) was investigated by Reclamation. The original alignment was found to have the shortest crest length, although explorations encountered thick (greater than 200 feet) deposits of overburden toward the south abutment. Subsequent explorations on the alternate alignment revealed that the overburden was at least as deep at the upstream axis. A determination of above ground embankment quantities showed that the alternate alignment resulted in about 10 percent fewer cubic yards, even though the crest length was longer and the dam higher (to get an equivalent reservoir storage). This appears to be because the original ground surface rises as one heads upstream. However, when evaluating the amount of below ground excavation required at both alignments, the longer axis at the alternate site resulted in significantly more excavation than the original site. When looking at total embankment quantities (all above and below ground fill materials), the alternate axis was estimated to have about 10 million more cubic yards than the original axis.

Since there were no obvious technical advantages (such as improved rock quality, better outlet works location, etc.), the large difference in embankment volumes, and resulting large cost increase, made it a relatively straightforward determination to select the original WIS site as Reclamation's preferred alignment.

Once the optimum axis was selected and the general type of embankment (rockfill dam) chosen, the next step was to further develop the two types of rockfill dams deemed most suitable to the Black Rock site. As mentioned earlier, these two alternatives are a concrete face rockfill dam and a central core rockfill dam. Both of these will be discussed in more detail in the following paragraphs. A plan view and typical sections of both alternatives are presented in Figures 43 and 44.

Concrete Face Rockfill Dam

General Design Concepts

One of the main advantages of a concrete face rockfill dam over any other type of embankment dam is that it does not contain a soil core vulnerable to erosion under a concentrated leak. The impervious element for this dam type is the upstream reinforced concrete face, which is not susceptible to erosion. This concrete face is tied into the rock foundation with a concrete plinth that acts as a thrust block or footing for the concrete face. Immediately downstream of the reinforced concrete face is a zone of sand and gravel with fines, which serves not only as a firm foundation for the concrete face slab, but also a key feature of the design. In the event of any leaks through the concrete face, a properly designed zone 2 forms a semi-pervious barrier that significantly increases head losses and thus reduces the amount of seepage. Thus, in the event of damage to the concrete face, whether from a failed waterstop or cracking induced by some type of differential settlement, seismic shaking or fault displacement, the zone 2 serves as an additional barrier to retard seepage.

A pervious transition, zone 3, is placed immediately downstream of the zone 2 and designed to be filter compatible with both the zone 2 and the downstream rockfill. In this way, should excessive flows occur through concentrated leaks, the zone 3 ensures that the zone 2 cannot erode and also provides sufficient drainage capability to handle seepage flows and allow them to pass into and through the large downstream rockfill section of the dam.

The rockfill zones are typically constructed in 3-foot thick lifts, and compacted with large vibratory rollers. The practice of spreading 3-foot lifts and then applying compaction tends to create a layer with larger rock at the bottom and an accumulation of fines at the top. Because of these stratified rockfill layers, it is widely accepted that the downstream rockfill will have high horizontal permeability and be able to drain off large leakage flows safely. This advantage is sometimes referred to as "flow-through capability of rockfill." A more detailed description of the various embankment zones, including expected material descriptions and construction procedures, are included in a later paragraph entitled "Embankment Zoning."

Crest Elevation

For all dam types being considered for the large reservoir size, the top of normal water surface (top of active conservation) is set at elevation 1775 to store 1.3 million acre-feet. The maximum reservoir water surface, assuming full storage of the Probable Maximum Flood above the normal water surface, corresponds to elevation 1778.

Freeboard heights were established using general rules, as well as checking with quick analyses. Because of the long reservoir and potential for high winds in the Black Rock Valley, wave runup will be a consideration at this site, as the combination of long fetch and high winds could create significant waves on the reservoir surface. The large reservoir option has a total reservoir length on the order of 10 miles, and it appears that wind gusts approaching 100 miles/hr are possible. A quick calculation of potential wave heights and runup confirmed that large waves would be possible. According to general guidance given in the Design of Small Dams [12], wave heights could be over 6 feet, and the suggested normal freeboard is 10 feet (about 1-1/2 times the wave height) for a typical dam with a riprap upstream slope. However, a different freeboard is required for a concrete face rockfill dam than for a rockfill dam with a rock upstream face. That is because the rougher surface of a rock face is much more effective than smooth concrete in dissipating wave runup. Design of Small Dams recommends providing 50 percent more freeboard if a smooth pavement is used on the upstream face. Consequently, the crest elevation for the concrete face rockfill dam will be 1790, providing a normal freeboard of 15 feet (as opposed to 10 feet for the earth core rockfill).

A quick check of expected deformations in the event of large ground motions possible at Black Rock Dam suggests that potential crest deformations would be only a few feet or less. Based on the seismic stability of a well constructed rockfill dam, extra freeboard in case of major embankment deformations does not appear to be required.

A general philosophy underlying the assignment of a 15-foot freeboard value is the belief that Black Rock Dam would not operate at full normal reservoir level very frequently. A full reservoir would likely require optimum conditions, not just in a particular water year but probably for a number of consecutive years. Thus, the average reservoir level in any given year will probably be below, and possibly well below, the top of active conservation level. Thus, the chances of a very large earthquake or flood occurring at the same time the reservoir is completely full is not judged to be very likely. What this means is that normally the freeboard will be greater than 15 feet.

Embankment Slopes

The crest width of Black Rock Dam will be 40 feet. Although wider than most dams, this width judged reasonable given the height of the dam and the high level of seismicity in the area. At this level of design, both the upstream and downstream slopes will be set at 1.5:1 (H:V). These are certainly not steep slopes for a concrete face rockfill dam, as some dams of this type have been built with 1.3:1 slopes, and a significant number have 1.4:1 slopes. However, considering the tall height, the high seismicity, and potential questionable areas of rock quality, these slopes appear justified. As the design progresses into future phases and more analysis is performed, steeper slopes and thus less material may be possible.

Thickness of Concrete Face

Recent design practice has been to have the concrete face thickness equal to around 1 foot (or slightly less) for dams less than 300 feet high, and for higher dams adding an incremental 0.002(H) thickness, where H is the total height of the dam. This means that the face slab will be 1 foot thick at the top of the dam and then gradually thicken at a constant rate of 0.2-foot for every 100 feet in dam height. Thus, at the deepest portion of the potential Black Rock Dam, the concrete face will be over 2 feet thick at the base of the dam.

Plinth Dimensions

The width (upstream to downstream) of the plinth is typically around 1/20 to 1/25 the height of the dam on hard rock foundations. Where rock quality is more suspect, plinth widths have been as wide as 1/10 the dam height. Since Black Rock Dam will have varying areas of rock quality, it is envisioned that the plinth width will vary in various portions of the foundation. For the purposes of an appraisal-level design and cost estimate, the plinth width will be designed to be approximately equal to 1/15 of the dam height. In areas of good rock and low dam height, the minimum width of the plinth will be set at 10 feet.

The thickness of the plinth is generally on the order of 1 to 1.5 feet, but in some cases reaches the thickness of the concrete face. At Black Rock Dam, it is envisioned that most areas of the plinth will range from 1 to 2 feet thick. For estimating purposes, the average thickness will be assumed to 1.5 feet.

Embankment Zoning

Since the concrete face serves as the impermeable membrane or water barrier of this dam type, the rest of the embankment consists primarily of rockfill. However, there are a couple of key zones immediately adjacent to the concrete face, as well as several zones comprised of materials from required excavation. These zones are shown on Figure 43 and discussed below.

Zone 1: This zone is comprised of any loessial materials that are excavated from the footprint of the dam. These fine-grained soils will be limited in extent, as they are believed to form a relatively thin (3 feet or less) mantle over much of the valley. These materials are to be separately stockpiled during excavation, and then placed along the toe of the concrete face as shown on Figure 43. As such, these materials may serve to fill in any crack in the concrete face should a significant fault offset occur during the life of the dam. These materials would be placed in 6-inch lifts and compacted by tamping rollers.

Zone 2: This is a processed, well graded sand and gravel zone, with fines, that serves a couple of key purposes. When compacted, this type of material serves as an excellent sub base for the concrete face. However, due to its well graded nature and fines content, it is not particularly permeable and serves to a certain extent as a second water barrier. In the event of cracks in the concrete face and resulting seepage passing through the face, this type of material should result in significant head losses. Typically, this material has a maximum particle size of 3 inches, and contains 45 to 65 percent gravel, 35 to 45 percent sand, and 2 to 12 percent fines. It is compacted by vibratory rollers. A secondary use of zone 2 material may be as a filter that is placed on areas of the bedrock foundation that are extensively weathered or perhaps fractured. As a filter, it would prevent piping of altered rock or underlying soil interbeds within the basalt.

Zone 3: This is a processed clean gravel and cobble zone, placed immediately downstream of the zone 2. It serves as a transition zone between the zone 2 and the rockfill, and also as a drainage element to control any flows that pass through the concrete face and zone 2. This zone will also be compacted by vibratory rollers. As with the zone 2, it may also be used as a foundation filter/drain in areas of questionable rock quality.

Zone 4: This is the basalt rockfill that forms the mass of the dam. It is envisioned to be quarried from the reservoir rims. Maximum size of the rock will be 3 feet. This rockfill will be placed in 3-foot lifts and compacted by large vibratory rollers, with moisture added as necessary.

Zone 5: This is a random fill zone comprised of the coarse-grained materials excavated from beneath the dam footprint. It will largely consist of Ringold sands, gravels, and cobbles. Because the properties and quality of these materials are expected to vary, this zone is embedded within the downstream portion of the rockfill, where it would have relatively the least impact on dam performance. These materials will be placed in approximate 2-foot layers and compacted to a dense state by large vibratory rollers. To achieve drainage through this layer (in the unlikely case drainage is required), periodic layers of zone 4 will be placed to ensure horizontal permeability.

Zone 6: This is a second random fill material comprised of the fine-grained materials excavated from beneath the dam. It is expected to consist mostly of lacustrine silt and clay layers within the Ringold Formation. This zone, in conjunction with zone 5, will comprise a portion of the downstream embankment, as well as serve as a relatively impermeable fill upstream of toe of the dam (refilling the excavation upstream from the concrete face and plinth. These materials will be placed in about 9-inch lifts and compacted to a dense state by tamping rollers.

Foundation Treatment

Because the concrete face and plinth are the key components comprising the water barrier of the concrete face rockfill dam, that is where the foundation treatment will be concentrated. Foundation treatment beneath the remainder of a rockfill dam is much less important, except in areas of highly weathered rock or fault zones where seepage/piping or displacement concerns exist. That type of special foundation treatment is discussed in a later section entitled "Additional Foundation Treatment for Embankment Dams". The amount of foundation treatment required in the upstream toe area will depend in large part on the quality of rock encountered. As discussed earlier, the width (as well as the depth) of the plinth will be adjusted as needed to accommodate rock quality, with a wider and perhaps deeper plinth in areas of poorer rock quality. In all areas, however, a minimum amount of treatment will be a combination of blanket (consolidation) and curtain grouting. Given the presence of fracturing in the basalts and areas of poor rock quality, extensive grouting is envisioned in certain areas. For this appraisal estimate, blanket grouting has been assumed for 30-foot depths and 7.5-foot centers throughout the plinth area. In addition, a multiple row grout curtain is envisioned, with depths ranging from 60 to 450 feet on 10-foot centers. For cost estimating purposes, a two-row curtain has been assumed and the average grout take for the entire grouting operation is assumed to be 2 sacks of cement per lineal foot of drill hole.

Central Core Rockfill Dam

General Design Concepts

One advantage of a rockfill dam with an earth core instead of a concrete face is that less damage may result in the event of fault offset within the dam footprint. Any type of fault offset would likely cause an area of extensive cracking in the concrete face. Although the rockfill dam would not be expected to fail, the reservoir may have to be drained after such an event and the concrete deck repaired. An earth core, by virtue of being more plastic or deformable, would be able to withstand fault offsets with a lesser degree of cracking. Furthermore, if the core contains appreciable clay, some healing of the cracks would be expected to occur. There is a distinct possibility that major repairs may not be required after a fault offset, or if repairs were required, they might include measures such as grouting, which would not entail draining the reservoir.

Whereas the concrete face rockfill dam relies on the concrete face as the water barrier, the barrier with this second alternative consists of an earth core comprised of relatively impermeable soils. An upstream sloping and relatively thin earth core was chosen for several reasons. The primary reason is that inclining the core upstream ensures that a large portion of the dam (the massive downstream zone) will consist of a strong, unsaturated rockfill, affording much static and dynamic stability. Secondly, the relative lack of impervious material available in the immediate area makes the core relatively expensive. Keeping this zone relatively thin is a means of minimizing costs to some extent. Additional cost savings are realized in a need for less foundation treatment, as the large zone of downstream rockfill needs far less foundation treatment than what is required beneath an impervious zone. Finally, inclining the core should help reduce the potential for the core to crack due to differing settlement properties of the rockfill and impervious material.

Immediately downstream of the earth core is a zone 2 filter zone, consisting of clean sand and gravel designed to be filter compatible with the zone 1 core thus preventing erosion of the core materials in the event of a crack. Downstream of the zone 2 filter is a clean gravel and cobble drainage zone to safely control and convey any seepage resulting from cracks in the core. The majority of the central core dam would be rockfill, as described above for the concrete face dam option. A more detailed description of the various embankment zones, including expected material descriptions and properties and construction procedures, are included in a later paragraph entitled "Embankment Zoning."

Crest Elevation

The selection of required freeboard has been described above under the concrete face rockfill dam alternative. To briefly repeat, for the large reservoir size, the top of normal water surface (top of active conservation) is set at elevation 1775 and the maximum reservoir water surface resulting from storage of the Probable Maximum Flood corresponds to elevation 1778. Because of the large fetch and potential for high winds, general guidance suggests a normal freeboard of 10 feet for an embankment dam with a rock face. Therefore, the crest elevation for the central core rockfill dam will be 1785 (five feet lower than for the concrete face dam), providing a freeboard of 10 feet.

Embankment Slopes

The crest width of a central core rockfill dam will be 40 feet, same as described for the concrete face rockfill alternative. Also similar to the concrete face dam, the downstream slope will be set at 1.5:1 (H:V). For the same reasons described for the concrete face alternative, this slope is judged reasonable, but may be able to be steepened during later designs. The upstream slope of the central core rockfill dam will be 1.75:1, slightly flatter than the concrete face dam. The flatter slope is to ensure stability of the upstream sloping central core.

Embankment Zoning

Although several of the zones in this rockfill dam are similar to the zones in the concrete face rockfill dam, there are some differences. The zones for the central core rockfill dam are shown on Figure 44 and discussed below.

Zone 1: This zone is significantly different from the zone 1 in the concrete face alternative (which was basically a random zone used at the upstream toe). For this central core rockfill dam, the zone 1 serves as the core, or water barrier, for the dam. As such, it is a critical zone and must be comprised of good materials. The ideal core material would be a clayey gravel, although a lean clay or silty gravel would also serve well. Because of the lack of such materials at the damsite, it is envisioned that these materials will need to be borrowed offsite. The zone 1 materials will be placed in 6-inch lifts and compacted to a dense state by tamping rollers. The moisture content of these soils will be carefully controlled to ensure that optimum properties for the core are achieved.

Zone 2: This is a processed, clean sand and gravel zone that serves as a critical filter for the zone 1 core. Although fairly similar to the zone 2 for the concrete face rockfill dam, this zone 2 will have a very low fines content. Because the zone serves as a filter, it is important that the material is as cohesionless as possible. This means that fines will be minimized, plastic fines not permitted, and any materials that display even a slight tendency toward cementation will be rejected. Zone 2 materials will be compacted by vibratory rollers. A secondary use of zone 2 material may be as a filter that is placed on areas of the bedrock foundation that may be extensively weathered or perhaps fractured. As a filter, it would prevent piping of altered rock or underlying soil interbeds within the basalt into the coarse rockfill.

Zone 3: This is a processed clean gravel and cobble zone, placed immediately downstream of the zone 2. It will be nearly identical to the zone 3 in the concrete face rockfill dam alternative. It serves as a transition zone between the zone 2 and the rockfill, and also as a drainage element to control any flows that pass through the concrete face and zone 2. This zone will also be compacted by vibratory rollers. As with the zone 2, it may also be used as a foundation filter/drain in areas of questionable rock quality.

Zone 4: This is the basalt rockfill that forms the mass of the dam. It is the same as described above for the concrete face rockfill alternative.

Zone 5: This is a random fill zone comprised of the coarse-grained materials excavated from beneath the dam. It is the same as described above for the concrete face rockfill alternative.

Zone 6: This is a second random fill material comprised of the fine-grained materials excavated from beneath the dam. It is the same as described above for the concrete face rockfill alternative.

Foundation Treatment

For the central core rockfill dam, foundation treatment measures will be concentrated beneath the zone 1 core of the dam (the water barrier). As described for the concrete face alternative, foundation treatment beneath the remainder of a rockfill dam is much less important, except in areas of highly weathered rock or fault zones where seepage/piping or displacement concerns exist. The amount of foundation treatment required beneath the core will depend in large part on the quality of rock encountered. To minimize the potential for stress concentrations and differential cracking, rock excavation and dental concrete will be used to shape the bedrock surface so as to minimize abrupt changes, overhangs, etc. In addition, slush grouting may be needed in areas where the rock is highly fractured or jointed and poses a risk of the zone 1 piping into such discontinuities. As with the concrete face alternative, a combination of blanket (consolidation) and curtain grouting will be utilized to improve rock strength and create a low permeability zone beneath the core. Given the presence of fracturing in the basalts and areas of poor rock quality, extensive grouting is envisioned in certain areas. For this Assessment, blanket grouting has been assumed for 30-foot depths and 10-foot centers over the entire footprint of the zone 1 core. In addition, a multiple row grout curtain is envisioned, with depths ranging from 60 to 450 feet on 10-foot centers. For cost estimating purposes, a two-row curtain has been assumed, and the average grout take for the entire grouting operation is assumed to be 2 sacks of cement per lineal foot of drill hole.

Additional Foundation Treatment for Embankment Dams

The previous discussions of the two rockfill alternatives have described anticipated foundation treatment measures beneath the impervious barriers. This section will describe additional foundation treatment measures applicable to both dams.

Overburden Excavation

As discussed under “Design Considerations,” it is difficult to determine at this level of design the amount of Ringold Formation to be removed. For this assessment study, two variations were developed, and the “average” excavation used in the cost estimate. The first option involved complete excavation to bedrock beneath the entire footprint of both rockfill dam alternatives. This will positively reduce all uncertainties of foundation liquefaction, and would also help support the use of steeper rockfill slopes in later designs. This option is shown on Figures 43 and 44 as “Complete Excavation to Bedrock.”

A second option is to excavate all of the overburden to bedrock beneath the upstream portions of both rockfill dam alternatives. This will ensure that the plinth and zone 1 core are founded on competent bedrock, and that foundation treatment and grouting can be effectively accomplished. In addition, for the concrete face alternative, it would serve to minimize the potential for settlement of overburden that may cause cracking of the concrete face slab. For most of the downstream portion of both rockfill alternatives, the foundation excavation would be taken down to competent Ringold materials, envisioned to be the compact and sometimes indurated gravels and cobbles indicated at depth in the drill logs. By removing the finer grained lacustrine deposits in the upper portions of the Ringold, this would minimize foundation settlements and the possibility of liquefaction of silt or sand layers. This option is shown on Figures 43 and 44 as “Excavation to Competent Ringold.” Either of these options may prove to be the best choice once additional information is learned about the properties of the Ringold. Since either option appears defendable at this stage, the cost estimates were based on the average of a total excavation of overburden and excavation to competent Ringold.

Localized Overexcavation of Rock

Due to the dipping nature of the basalt flows resulting from the folding in the Black Rock Valley, several different basalt flows, as well as sedimentary interbeds, will be encountered during foundation excavation. The quality of rock at the contacts of these various flows is expected to be poor, and localized overexcavation to remove poor quality rock is anticipated. In addition, there will likely be some areas where only a thin veneer of basalt exists over an interbed. An example of this occurs in WIS drill hole DH-3 in the upper left abutment, where only about 22 feet of basalt overlies 33 feet of a silty clay interbed zone. In a case like this, depending on the height of the overlying embankment, it may be prudent to remove both the thin veneer of basalt and the interbed zone to found the embankment on a thicker and more competent basalt unit. This would minimize settlement, seepage, and potentially even liquefaction concerns.

Right Abutment Fault Treatment

There is insufficient information at this stage to positively confirm the presence and particularly the nature of the apparent fault beneath the right abutment. Such a fault zone could consist of a thick clayey gouge zone or perhaps a wide area of extensive shearing of rock units. A clayey fault zone might be fairly impermeable but might still be vulnerable to piping considering the high hydraulic heads due to this large dam. A highly fractured zone would raise issues of extensive seepage at the base of the right abutment. A combination of clay and fracturing might raise a concern about the clay piping into the fractured rock. Either clayey or fractured materials may not provide an ideal foundation for a plinth or even a clay core.
The types of treatment considered for any fault zone uncovered will depend on the nature of the material exposed. Soft materials will require additional excavation to ensure that the plinth or core can be founded on a relatively firm foundation. Fractured rock will likely require an increased amount of blanket and curtain grouting beneath the plinth or zone 1 core. In addition, a highly permeable fault zone may require the placement of impermeable soil materials upstream of the water barrier which would serve as an upstream blanket to reduce seepage and increase head losses. Downstream of the plinth or zone 1 core, such a pervious zone would likely be covered by a two-stage zone 2 and zone 3 filter/drain to minimize the potential for piping of any deteriorated rock or embankment materials into the rockfill shells. A very deep fault zone may even require some type of cutoff such as a secant pile wall to depth.

Miscellaneous Bedrock Treatment

Special foundation treatment downstream (and perhaps upstream) of the plinth or the zone 1 core will be required in areas of particularly poor rock quality, which may include highly fractured rock, highly weathered or altered rock, or areas of faulting. In such locations, filters may need to be placed downstream of the plinth or core for a distance of about one-quarter of the water head. (If severe fracturing was encountered, perhaps a lean concrete or shotcrete blanket would first be placed on the foundation before filter placement.) The filters would consist of two stages, similar to zone 2 and zone 3 used behind the concrete face and zone 1 core. This method is envisioned to prevent any potential piping of poor foundation materials (particular fault gouge or weathered rock) into the coarse rockfill embankment. Potential upstream treatment in areas of faulting or highly fractured rock might be to locally increase the width of the plinth or core, perform additional grouting, or even place an impervious blanket for a distance upstream of the plinth or core.

Small Embankment Dams

Design Considerations

The same design considerations that were discussed for the larger dam alternatives also apply to the smaller dam alternatives.

Concept Description

The dams impounding the small reservoir are still very large embankments and the previous discussions of the large reservoir options also apply to the small dams. For all dam types being considered for the small reservoir size, the top of normal water surface (top of active conservation) is set at elevation 1707.0 to store 800,000 acre-feet. The maximum reservoir water surface, assuming full storage of the Probable Maximum Flood above the normal water surface, corresponds to elevation 1712.0. The only difference between the large and small dams is their crest elevation. A comparison of the large and small embankment dams is shown in Table 14 below.

Table 14. Comparison of Large and Small Embankment Dams

Alternative	**Crest Elevation (feet)**	**Crest Length (feet)**	**Dam Height* (feet)**
Concrete Face Rockfill – Large Reservoir	1790	6,615	760
Concrete Face Rockfill – Small Reservoir	1722	6,255	692
Central Core Rockfill – Large Reservoir	1785	6,590	755
Central Core Rockfill – Small Reservoir	1717	6,220	687

*Note: Dam height is the maximum structural height (from crest of dam to bottom of foundation excavation)

A plan view and typical sections of both small dam alternatives are presented in Figures 45 (concrete face rockfill dam) and 46 (central core rockfill dam).

Large Roller Compacted Concrete (RCC) Dam

Design Considerations

Roller Compacted Concrete (RCC) is a no-slump concrete that is placed by earth-moving equipment and compacted by vibrating rollers in horizontal lifts up to 12 inches thick. The materials used for RCC are the same as those used for conventional mass concrete and include water, cementitious materials (cement and pozzolan), admixtures, and fine and coarse aggregate. RCC was selected for the concrete dam alternative instead of mass concrete because it is more economical for wide canyons. The upstream and downstream faces of the dam would be slip-formed conventional concrete that would serve as forms for the RCC placement. Crack inducers and waterstops would be placed to form contraction joints. The dam would have a drainage gallery, and formed drains would be drilled from the top of the dam to the gallery located near the upstream face to intercept leakage through the lift lines.

Concept Description

The dam crest elevation was established to contain the maximum reservoir resulting from storing the PMF with six feet of freeboard to the top of the parapets. This set the top of dam at elevation 1781.0 and top of parapets at elevation 1784.0. The crest width would be 20 feet, which is typical for RCC gravity dams, and the downstream slope would be 0.75:1 starting at the downstream edge of the crest. A cursory stability analysis run to validate that the downstream slope of 0.75:1 provided sufficient weight for stability identified that internal drains would be important for stability. Bonding mortar at each lift line was included in the estimate based on the large surface area and assuming that the preferred "8 hour cure time" between lifts would likely not be met. A plan view and typical sections of the large RCC dam alternative are presented in Figures 47 and 48.

Contraction joints were located at 50 foot spacing and waterstops were incorporated into the upstream face of the dam at all joints. Galleries were located near original ground to provide drainage by gravity flow. The formed drains into the galleries were assumed to be spaced at 10-foot centers.

A cement content of 275 lbs/yd^3 with 40 percent fly ash and 60 percent cement was selected for the RCC mix. Aggregate was assumed to be hauled from either the Columbia River or Yakima River area due to the anticipated high cost of on-site processing of the Ringold Formation. An RCC test section was assumed to be constructed which would later be used as a thrust block for the outlet works.

Foundation Treatment

The RCC dam requires a competent rock foundation under the entire footprint of the dam. Foundation excavation was assumed to the top of competent rock based on available geologic data which results in 200 feet of excavation towards the right abutment. It was assumed that 10 feet of rock would need to be removed from the overall footprint to reach competent material.

Foundation drainage and grouting was patterned in accordance with standard Reclamation criteria. Adits were assumed into the right abutment for grouting and drainage of the shear zone and foundation grout takes was assumed to be 2 sacks of cement per lineal foot of drill hole.

Small Roller Compacted Concrete (RCC) Dam

The small RCC dam is still a very large dam and previous discussions of the large reservoir option also apply to the small reservoir option. The only difference is the top of dam will be set at elevation 1715.0 and top of parapets at elevation 1718.0. A comparison of the large and small RCC dams is shown in Table 15 below. A plan view and typical sections of the small RCC dam alternative are presented in Figures 49 and 50.

Table 15. Comparison of Large and Small RCC Dams

Alternative	Top of Dam Elevation (feet)	Crest Length (feet)	Dam Height* (feet)
Large Reservoir	1781	6,695	751
Small Reservoir	1715	6,200	685

*Note: Dam height is the maximum structural height (from top of dam to bottom of foundation excavation)

Spillway

During the technical site review (Appendix A), the project team viewed the WIS proposed saddle dam/spillway location on the south side of the reservoir. Although the saddle dam/spillway location appeared feasible, there was concern that significant channel improvements including modification or replacement of an existing bridge on State Highway 241 would be required to safely carry any spillway discharge to Dry Creek. There is also some concern about the environmental impacts of channeling discharges from Black Rock Reservoir into Dry Creek that would eventually find their way to the Yakima River. Because of these concerns, Reclamation investigated the possibility of storing the Probable Maximum Flood (PMF) in the reservoir instead of constructing appurtenant structures to handle the flood. The fact that this is an offstream storage reservoir with a large surface area, led to the decision to raise the dams to store the PMF volumes identified in the Hydrologic Hazard section of this report (Appendix C), thus eliminating the need for a spillway. For the large dam, the increase in height for storage of the winter PMF in the reservoir was 3 feet. For the small dam, the increase in height for storage of the winter PMF was 5 feet.

Low Level Outlet Works

The low level outlet works is a dam safety feature that will evacuate the reservoir in the event of an emergency, spilling flows into the normally dry Dry Creek. The outlet works were sized to meet Reclamation's reservoir evacuation criteria outlined in ACER Technical Memorandum No. 3 [13]. Trashracks were provided at the river outlet works intakes; however, fish screens were not provided since this outlet would be used infrequently for emergency evacuation only. No separate detailed designs were prepared for the outlet works for the small reservoir, but rather, quantities were reviewed and a judgment made that costs for the small reservoir outlet works options would be about 95 percent of the costs of the outlet works for the large reservoir options.

Outlet Works for Embankment Dams

The outlet works for the embankment dams was located on the left (north) abutment as opposed to the right (south) abutment because the right abutment has what is perceived to be a significant shear zone. The right abutment would result in a shorter distance, however tunneling through this material was considered too risky without additional data. Concrete thicknesses for the conduits were based on other Reclamation projects with features of similar sizes. The upstream conduit was steel-lined due to extreme pressures and potentially weak geology (fractured rock). Steel pipe thicknesses were sized to handle full reservoir pressures. The downstream steel pipe was assumed to be buried for support in lieu of using concrete or steel structural support. An emergency fixed-wheel gate housed in a gate chamber at the bottom of a vertical shaft was selected to reduce the length of the pressure pipe through the dam. See Figures 51 and 52 for plans and sections of the proposed low level outlet works for the large and small embankment dams respectively.

The outlet works discharge is controlled by two 108-inch diameter jet flow gates. These gates were chosen and sized based on velocities which could be safely tolerated without special fabrication requirements. In consideration of the anticipated limited use of the outlet works, a plunge pool stilling basin, lined with impervious material and riprap, was selected in lieu of a conventional concrete stilling basin to reduce costs. The basin size was estimated using equations currently being developed via a research project with Reclamation's Water Resources Research Laboratories. Large concrete thrust blocks are required and were sized with a factor of safety of 2.0 to handle anticipated thrust loads at the pipe bends.

Outlet Works for RCC Dam

The intake for the outlet works for the RCC dam alternative was located on the upstream face of the dam near its right abutment. Because the outlet works would be constructed within the dam structure itself, it was located near the right abutment to reduce its length. The outlet works intake structure would have trashracks and an emergency fixed-wheel gate. The downstream steel pipe was assumed to be buried for support in lieu of using concrete or steel structural support. The outlet works discharge is controlled by two, 108-inch-diameter jet flow gates. In consideration of the anticipated limited use of the outlet works, a plunge pool stilling basin, lined with impervious material and riprap, was selected in lieu of a conventional concrete stilling basin to reduce costs. Large concrete thrust blocks are required and were sized using a safety factor of 2.0 to handle anticipated thrust loads at the pipe bends. See Figures 48 and 50 for plans and sections of the proposed low level outlet works for the large and small RCC dams respectively.

Highway and Utility Relocations

The proposed Black Rock Reservoir will inundate up to 13.5 square miles of Black Rock Valley including State Highway 24 (SH24), a buried fiber optic line along SH24, and two overhead 115-kV lines on H-frame type wood-pole supports. For this Assessment, SH24 was assumed to be relocated from the valley floor and the transmission lines removed and routed along the new SH24 alignment. The existing buried fiber optic line would be abandoned in place and a new line would be run along the new SH24 alignment.

Design Considerations

The WIS Final Report [2] relocated SH24 to the south of the reservoir in the Rattlesnake Hills and indicated that residents of Black Rock Valley would prefer a northern relocation. For this study, Reclamation also relocated SH24 to the south of the reservoir because topography on the north side of the reservoir is not conducive for this road relocation. A north side valley alignment would require several bridges to span over existing draws or the road would need to be constructed on land currently within the Yakima Firing Center Military Reservation. The bridges would add significant cost to the road relocation and construction on the firing center raises security concerns and the possibility of encountering unexploded ordinance during construction.

Concept Description

The selected alignment for the relocated SH24 is similar to the alignment in the study by WIS [2], however, since the aerial topography provided for this study is more detailed than the USGS topographic maps that were used by WIS, a more refined horizontal alignment and vertical profile were defined. The alignment was adjusted in order to avoid some of the difficult terrain encountered and to improve the crossing over Rattlesnake Hills. In addition, the alignment was straightened and larger horizontal curves were included in order to accommodate a 50-70 mph speed limit. Also, the vertical profile was adjusted in order to follow the existing terrain and to better balance cut and fill volumes. Even with these modifications, large cut and fill areas were encountered along the proposed alignment which significantly affected the estimated cost of this road relocation.

The selected alignment consists of approximately 11.8 miles of relocated highway. See Figures 53 and 54. The road cross-section is typically 40-feet wide with two 12-foot lanes and 8-foot wide shoulders on each side, and consists of a 9-inch base course and 6 inches of asphalt concrete surfacing. Guardrail is provided on either side of the road when the height of embankment exceeds 10-feet. The width of the road is increased by at least 4 feet when guardrail is provided. Therefore, when guardrail is provided on both sides the road width is increased to 48 feet minimum. The horizontal alignment has a minimum road centerline radius of 4,000 feet, and the vertical alignment uses a maximum grade of 7 percent. The fill and cut slopes vary from 6:1 to 2:1 depending on the height of the cut or fill. The design speed is 70 mph for level terrain, and 50 mph for mountainous terrain.

Construction Considerations

Foundation treatment: The potential for varying rock quality and faults within the foundation for Black Rock Dam will necessitate a flexible working relationship with the contractor. Additional excavation will be required in some areas while additional treatment measures such as dental concrete, slush grouting, and filter blankets, will be required in other areas. These locations can not be identified on design drawings and will need to be determined during construction.

Depth of Ringold excavation: If the final design founds part of the dam on competent Ringold Formation, the field staff and designers will need to carefully evaluate the surface of the exposed Ringold and possibly conduct explorations during construction in order to determine a suitable depth of excavation. It is envisioned that only the lower portion of Ringold will provide a satisfactory foundation for much of the embankment.

Embankment compaction: Due to the high seismicity, it will be critical to ensure that all embankment zones are compacted to maximum practicable densities in order to preclude liquefaction. Close inspection and testing will be necessary to ensure proper moisture contents and densities are being achieved.

Random fill zones: As shown on the drawings, a large random fill zone will be located within the downstream portion of the rockfill embankment to utilize materials from required excavation. It is anticipated that these materials will vary widely in composition. These materials will be excavated and stockpiled separately into two general categories – fine-grained and coarse-grained materials. The finer grained materials will be placed in thinner lifts and compacted by tamping rollers while the coarser materials will be placed in thicker lifts and compacted by vibratory rollers. As both excavation/stockpiling and fill placement operations proceed, careful attention will need to be paid to ensure that these random fill materials are properly classified, moisture control is optimized, and the proper method of compaction is used to ensure a thoroughly compacted zone.

Staged construction: To gain additional knowledge of the site prior to issuing a full contract, as well to optimize scheduling of the construction work, a staged construction could be considered. A first stage could include foundation excavation and stockpiling, and possibly foundation grouting. A second stage would include the bulk of the earthwork placement.

Highway Relocation: Detours should not be necessary as long as the existing SH24 can be left in place during the construction of the relocated highway alignment. Disruptions to traffic on SH24 and SH241 will occur when the relocated alignment is connected to the existing highways. The excavation was assumed to be mostly in bedrock and therefore drilling and blasting will be required.

VIII. Outflow Conveyance System

Design Considerations

During their reconnaissance study, WIS investigated several options to deliver water from Black Rock Reservoir to the Roza Canal and all delivery options included a multi-level intake to selectively withdraw water from the reservoir. For this assessment study, outflow conveyance options that deliver water to the Roza Canal at MP 22.6 (near the intersection of SH24 and Roza Canal) were considered. A design flow capacity of 2,500 cfs was selected for the outflow conveyance based on the assumption of providing the Roza and Sunnyside Divisions' entire water supply from the Columbia River in lieu of Yakima River. This amounts to an instantaneous flow of about 2,362 cfs plus an allowance for other entities whose main conveyance facilities are in the vicinity of the Roza Canal.

Hydraulic Design

The initial outflow conveyance system from Black Rock Reservoir to the Roza Canal investigated during this Assessment consisted of an 18-foot-diameter tunnel and a 15.5-foot-diameter pipeline through Moxee Valley along SH24. For this alignment, the best surge tank site was located where the conveyance was still in basalt bedrock, about 10 miles from Black Rock Reservoir, and about 8 miles from Roza Canal. However, transient analyses of this arrangement indicated that the surge tank was located too far from the proposed Black Rock Powerplant at Roza Canal to prevent negative downsurge pressures from occurring in the pipeline and tunnel when the wicket gates on the Francis turbine closed in 60 seconds or less. Attempts to mitigate the negative pressures by slowing down wicket gate closure times led to unreasonable gate closure times near 10 minutes.

To maintain the valley alignment and prevent negative downsurge pressures, use of a synchronous bypass with the Francis turbine, or use of a Pelton turbine in lieu of the Francis turbine were considered. The use of a synchronous bypass where a valve opens at the same time as the wicket gates close upon load rejection could minimize and/or eliminate the negative downsurge pressure. However, the design would rely on a mechanical and/or electrical means to open the valve. While this may be technically feasible, Reclamation's current practice is to not rely on mechanical devices to prevent detrimental transient pressures because we consider the risks associated with a synchronous bypass failure (valve failure and/or valve reaction lag time) to be unacceptable.

Another method to mitigate the downsurge pressures is through the use of a Pelton turbine in lieu of the Francis turbine. Pelton turbines are easier to put into a system with transient problems because upon load rejection, deflectors aim the water away from the turbine wheel, maintaining flow without prolonged overspeed conditions which are harmful to the generator. The flow through the Pelton turbine can thus be reduced very slowly, preventing or at least lessening high transient pressures. However, a Pelton turbine is not suitable for the low head acting on the outflow conveyance system. At low reservoir water surface elevation of 1500 feet, the static head, without including friction loss through the tunnel and pipe is 330 feet. The heads typically required for Pelton turbines to be economical are approximately 500 feet and higher.

A canal through the Black Rock and Moxee Valleys was also investigated but topographic features in these valleys preclude the economic use of a canal to deliver water to the Roza Canal. On the west side of Black Rock Reservoir, the ground is at approximate elevation 1800.0 feet and then slopes gradually down to Roza Canal. In order to make deliveries at low reservoir, elevation 1500 feet, a long tunnel would be needed to convey water to a point in the Moxee Valley where the ground is at or below elevation 1500 feet. This location is close to the Roza Canal so the canal length would be small compared to the tunnel length and any cost savings from canal construction would be offset by the tunnel construction. In order to reduce the length of the tunnel and increase length of canal, a pumping plant would be required to lift water from Black Rock Reservoir over the high point between the Black Rock and Moxee Valleys

In the end, Reclamation decided to reduce the distance between the powerplant and surge tank to mitigate the downsurge pressures. This required moving the outflow alignment out of the valley floor. A route following the southern edge of the Yakima Ridge Mountains was analyzed and found to work for the pressures and flows of the outflow system. The tunnel and pipeline diameters were sized using the maximum flow of 2,500 cfs and turbine flow of 1,500 cfs as explained in the Black Rock Outlet Facility section of this report. A Black Rock Reservoir elevation of 1500.0 feet (Top of Inactive) was used to calculate the maximum downsurge, to size the tunnel and pipeline diameters, and to size the surge shaft. A Black Rock Reservoir elevation of 1775.0 feet (Top of Active) was used to calculate the maximum pressure at the powerplant and to determine the elevation required at the top of the surge shaft. The required minimum water surface, or hydraulic grade line at the Black Rock Powerplant was established at elevation 1364.0 feet. This hydraulic grade line enables the water to be delivered to the Sunnyside Powerplant without the need for an intermediate pumping plant.

To accommodate surge shaft requirements, the outflow conveyance system was aligned as shown in Figures 55 and 56. The plan and profiles are based on USGS topographic maps because this alignment is outside the contour data developed from the aerial photogrammetry. The 17-foot-diameter tunnel begins southeast of Taylor Ranch on the north side of Black Rock Reservoir and parallels the southern edge of Yakima Ridge for approximately 14 miles to reach the 40-foot-diameter surge shaft location. After the surge shaft, the tunnel angles out of the mountains and ends on the north side of SH24 where it transitions to a 17-foot-diameter buried steel pipe that crosses under SH24 and runs to Black Rock Powerplant. The distance from the surge shaft to the powerplant for this alignment is approximately 19,000 feet (about 3.5 miles).

The following factors and assumptions were used for the hydraulic and transient analysis of the 2,500 cfs outflow conveyance system:

Tunnel Design Flow:	2,500 cfs
Turbine Design Flow:	1,500 cfs
Colebrook White Rugosity Value:	0.002
Downsurge pressures:	No negative pressures.
Required HGL for Sunnyside Diversion:	El. 1364.0
Wicket gates closing time:	60 seconds

The tunnel was sized using the Colebrook and White Rugosity value of 0.002 as listed in Reclamation's Design Standards No. 3 [11] for concrete lined tunnels and checked using the Hazen Williams equation and a coefficient of 120. The hydraulic grade line at Roza Canal for a 17-foot-diameter tunnel and steel pipe at the design flow of 2,500 cfs is approximately elevation 1368 at minimum reservoir water surface elevation of 1500 feet.

A transient analysis was performed using Reclamation's Transient Analysis for Pipe Systems (TAPS). The transient analysis showed that at the full design flow of 2,500 cfs, large negative downsurge pressures would occur if a surge relief device was not within 2,000 feet of the powerplant. However, using the turbine design flow of 1,500 cfs, the surge relief point could be farther upstream from the powerplant and the distance of 19,000 feet between surge shaft and powerplant was determined to be acceptable. The surge shaft diameter was set at 40 feet to prevent dewatering and the top of the surge shaft was set at elevation 1900 feet to prevent overtopping during a unit load rejection. Both the high and low Black Rock Reservoir elevations were analyzed to get the worst-case upsurge and downsurge conditions, respectively. The maximum hydraulic grade line at the powerplant was determined to be elevation 1950 feet which converts to 810 feet of pressure based on a pump/pipeline centerline elevation of 1140 feet. See Figure 57 for a graphical representation of the hydraulic grade lines for the Outflow Conveyance System. Transient analyses using a turbine flow of 900 cfs were not completed during this study.

Concept Description

In Black Rock Reservoir, a single-level intake at elevation 1500 feet was sized for the outflow conveyance system. A multi-level intake was not considered for this study because no specific water quality objectives have been identified for the irrigation water and there are no downstream fish water quality considerations. Fish screens were included on the outlet structure to prevent fish that may be stocked in the reservoir from migrating into the Yakima Basin. Fish screen sizing criteria was assumed to be the same criteria used to size the intake structure at Priest Rapids Reservoir. The fish-screened intake assumed for this study is a half-cylinder shaped screen supported on a reinforced concrete slab. An air burst backwash system is included for cleaning of the screens and bulkhead gates and guides are included for dewatering the outflow conveyance system during emergencies.

The outflow tunnel design considerations are similar to the inflow tunnel design considerations noted in the Inflow Conveyance section of this report. The steel pipe earthwork quantities between the downstream tunnel portal and Black Rock Powerplant were calculated based on the Outflow Pipe Trench Section shown on Figure 56. At the deep excavation for the downstream tunnel portal, the earthwork quantities assumed 10 foot benches every 20 feet in depth.

Construction Considerations

Pipe Fabrication: Steel pipe sections 16-feet in diameter and greater will have to be transported in pieces by truck or rail and welded together and pressure tested in the field.

Tunnel Excavation: The potential for varying rock quality encountered during tunnel excavation will necessitate a flexible working relationship with the contractor. Differing ground, water and gas conditions from those anticipated will affect the tunnel construction. Based on current knowledge of the tunnel alignment, bedrock will consist of a series of basaltic lava flows and associated interflow sediments. The basaltic portion of the tunnel will be in rock which will likely vary structurally and texturally from massive nonvesicular to highly vesicular and flow-breccia types. Soft, relatively uncemented interflow sediments will also likely be encountered. A number of shear zones, consisting of fractured rock with soft gouge materials may also be encountered during tunneling.

IX. Black Rock Outlet Facility

The proposed Black Rock Outlet Facility is located adjacent to the Roza Canal (MP 22.6) on the southeast corner of the Roza Canal and SH 24 intersection. The facility includes a powerplant, bypass structure to permit water deliveries when the unit is not on line or to pass flows in excess of powerplant design flows, flowmeter, and manifold piping and valving for pressure pipe diversions to the Roza and Sunnyside Irrigation Districts. Reclamation selected this location based on its position within the Roza and Sunnyside delivery systems, proximity to Black Rock Reservoir, and ease of access from SH24. See Figures 1 and 60.

Design Considerations

Two powerplant design flow options were developed to account for at least two of the possible delivery scenarios to the Roza and Sunnyside Irrigation Districts. Option 1 would pass all water from Black Rock Reservoir through the powerplant and/or bypass structure into a modified Roza Canal before being delivered to Roza and Sunnyside water users. Option 2 would require a smaller capacity powerplant because bifurcations upstream from the plant would provide pressurized pipeline water deliveries to Roza water users north of the delivery point, and all Sunnyside water users. Although the peak design flow of the outflow system is 2,500 cfs, the powerplant for Option 1 was designed for a flow of 1,500 cfs, and the powerplant for Option 2 was designed for a flow of 900 cfs. This was done so that the plants could be operated at full capacity for most of the irrigation season (April through October) and to reduce equipment costs. Reclamation selected a 1,500 cfs turbine design flow for Option 1 based on providing the Roza and Sunnyside District's entire water supply from the Columbia River in lieu of Yakima River and it is representative of their combined April Yakima River water rights. The 900 cfs turbine design flow was selected for Option 2 assuming water for Sunnyside and Roza water users north of MP 22.6 would be diverted upstream from the powerplant and is representative of anticipated deliveries to Roza water users south of MP 22.6 (885 cfs). The bypass structure for both options was sized to pass the total outflow design flow of 2,500 cfs. The two distribution scenarios are shown schematically in Figures 58 and 59. Other combinations of one district's water being delivered to the canal while the other district's water is bifurcated upstream from the powerplant are possible but were not evaluated during this study. Future power operations studies should be conducted during the feasibility design to better define the rated capacity of the plants.

Black Rock Reservoir operating water surface elevations range from a low of 1500 feet to a high of 1775 feet. The water surface elevation in the Roza Canal at MP 22.6 was assumed to be approximately 1170 feet. The steady state head at the Black Rock Outlet Facility (measured from canal water surface) ranges from a low of 198 feet to a high of 477 feet. The powerplant design head for turbine sizing was assumed to be the average of the steady state head. Details and quantities for the 1,500 cfs powerplant and 2,500 cfs bypass structure were developed during this study but no detailed layout or transient study of the 900 cfs powerplant was prepared. For the 900 cfs powerplant option, the bypass was conservatively sized for the full outflow capacity to provide a means of bypassing the powerplant and pressurized water deliveries. This sizing should be reviewed and revised as necessary during the feasibility design. Quantities used to estimate the cost of the 900 cfs powerplant and 2,500 cfs bypass structure were obtained by reducing the excavation and concrete quantities for the reduced unit submergence and adjusting major mechanical items. A comparison of the 1,500 cfs and 900 cfs plants is shown in Table 16.

Table 16. Black Rock Powerplant Unit Data

Unit Data	1,500 cfs Powerplant	900 cfs Powerplant
Number/Type of Units:	1 Francis	1 Francis
Design Discharge:	1,500 cfs	900 cfs
Design Head:	338 feet	338 feet
Speed:	327 rpm	400 rpm
Assumed Unit Efficiency:	90 percent	90 percent
Power Output at Design Head:	38 MW	23 MW
Min. Turbine Submergence	30.5 feet	25 feet
Max. Scroll Case Dimension	23.5 feet	19.0 feet
Bottom of Draft Tube	El. 1118.8	El. 1128.3
Turbine Guard Valve:	108-inch spherical	84-inch spherical

Concept Description

The Black Rock Powerplant is an indoor type plant with a structural steel superstructure enclosed with concrete masonry walls. The intermediate and substructure are reinforced concrete. The powerplant consists of a service bay and a single unit bay. The powerplant and bypass structure share the same superstructure but are separated structurally by an expansion joint. The Bypass Structure houses four 84-inch sleeve valves to dissipate head. Each sleeve valve discharges into a 33-foot-diameter by 20-foot high stainless steel-lined stilling chamber. One 90-ton overhead traveling crane is provided to handle the powerplant and bypass electrical and mechanical items. See Figures 61 though 65 for general arrangements of the powerplant and bypass structure.

The guard valves selected for the turbines and sleeve valves are spherical valves. Spherical valves are fully ported valves designed for high velocities and high pressures. Because they are designed for high velocities, smaller valve diameters can be used in larger diameter pipe. Because of the large valves needed for these facilities, they would most likely be shipped in parts with the final assembly and testing being done in the field.

Steel Piping for the tunnels, manifolds, and penstocks was designed in accordance with AWWA M11 [9] and ASCE Manuals and Reports on Engineering Practice No. 79 [10]. The large diameter tunnel liners and high pressure manifolds were designed with ASTM A572 Grade 50 steel plate. All other manifolds, penstocks, and outlet works piping were fabricated from ASTM A36 steel plate.

A concrete-lined, open-channel outlet transition structure was sized to convey the outlet flows into the Roza Canal. The service yard was sized to permit mobile crane access around the structures and a 7-foot chain link fence was provided around the yard for security. See Figure 60.

The Black Rock Powerplant utilizes standard 5 kV non-segregated bus and vacuum type unit circuit breakers. Incoming power was assumed to be from a tap on the Reclamation power line about 1,000 feet from the Black Rock Outlet Facility. For this Assessment, Reclamation assumed that this is a 34.5-kilovolt line that originates at the existing Roza Powerplant switchyard. The line tap will be a wood-pole tap structure. A 75-foot by 100-foot switchyard is located within the outlet facility service yard. The switchyard will include a transformer, circuit breakers, and disconnect switches. The use of SCADA equipment was not included in the study.

Construction Considerations

Canal Bypass: The need for a temporary canal bypass was assumed in Reclamation’s estimates. Upstream and downstream earthen cofferdams with geomembrane linings would be constructed in order to connect the transition structures to the canals. Three 9-foot-diameter corrugated metal pipes between the cofferdams would permit canal operation during construction.

X. Delivery Systems

Roza Division Delivery System

Options to deliver Black Rock water from MP 22.6 of the Roza Canal to water users within the Roza Irrigation District were evaluated by Reclamation's Pacific Northwest

Construction Office. The results of their evaluation are documented in the report entitled Preliminary Assessment of Black Rock Delivery System for Roza, Terrace Heights, Selah-Moxee, and Union Gap Irrigation Districts [3]. At the Black Rock Outlet Facility location, 215 cfs of water delivery is required to Roza water users located upstream (north) from MP 22.6, and 885 cfs of water delivery is required to Roza water users located downstream (south) from MP 22.6. Upstream deliveries are proposed to be made by one of two methods: 1) Bifurcation of a steel pipe off the outflow conveyance pipe upstream of the new Black Rock Powerplant, or 2) Pass all Roza water through the Black Rock Powerplant and bypass structure and construct a canal-side pumping plant to lift water to a new high pressure distribution pipeline to supply the Roza-North water users.

Sunnyside Division Delivery System

Options to deliver Black Rock water from MP 22.6 of the Roza Canal to water users within the Sunnyside Irrigation District were evaluated by Reclamation's Pacific Northwest Regional Office. The results of this evaluation are documented in the report entitled Preliminary Assessment of Black Rock Delivery System for Sunnyside Division [4]. Two methods of delivery were evaluated:

- Option 1: Bifurcation of a steel pipe off the outflow conveyance pipe upstream of the new Black Rock Powerplant.

- Option 2: Pass all Sunnyside water through the Black Rock Powerplant and bypass structure and modify the Roza Canal for the higher flows.

Sunnyside Division Delivery Option 1 – Hydraulic Design Considerations

For this option, a pipe would bifurcate off the outflow conveyance pipe directly upstream of the new Black Rock Powerplant. This pipeline is a continuation of the outflow conduit from Black Rock Reservoir and will be subject to the pressures associated with the reservoir fluctuation. The alignment begins by generally following the Roza Canal across orchards to the top of Konnowock Pass, and then following Konnowock Pass Road down to Sunnyside Canal. At the Sunnyside Canal, a powerplant and bypass structure would be constructed to dissipate the excess 435 feet of head. The pipeline diameter was sized using a flow of 1,262 cfs, while the design of the turbine was based on a flow of 900 cfs. This was done so that the plant could be operated at full capacity for most of the irrigation season (April through October) and to reduce equipment costs. The 900 cfs turbine design flow was selected based on providing the Sunnyside District's entire water supply from the Columbia River in lieu of Yakima River and is representative of their April Yakima River water rights.

The Black Rock Reservoir elevation of 1500.0 feet (Top of Inactive) was used to calculate maximum downsurge pressures, and the Black Rock Reservoir elevation of 1775 feet (Top of Active) was used to calculate the maximum pressure at the powerplant. The following factors were used for the hydraulic and transient analysis for the Sunnyside Pipeline:

Pipeline Design Flow:	1,262 cfs
Turbine Design Flow:	900 cfs
Hazen Williams Coefficient:	135 (Friction Loss)
Pipeline Diameter:	12 feet
Downsurge pressures:	No negative pressures.

The transient analysis of the Sunnyside Powerplant and pipeline from Black Rock Powerplant showed that closing the turbine wicket gates at Sunnyside Powerplant in 60 seconds would result in negative downsurge pressures at approximate Station 206+00. In order to keep the downsurge pressures from being negative, the wicket gate closure time was increased to 68 seconds. Other options that were evaluated to address the negative pressures are shown in Table 17.

Table 17. Sunnyside Pipe Options

Option	Diameter feet	Turbine design flow cfs	Wicket gate closure time seconds
1	12	900	68
2	12	700	60
3	14	900	60

The maximum elevation of the hydraulic grade line at the pumping plant was determined to be El. 2144 feet which converts to 1,272 feet of pressure based on a pump/pipeline centerline elevation of 872 feet. The effects of the Sunnyside transient pressures at Black Rock Powerplant and the rest of the system are less than the transient pressures generated by Black Rock Powerplant. Future Studies should analyze the effects of both powerplants shutting down at the same time.

The use of in-line generators to reduce head in the Roza and Sunnyside pressure delivery systems was considered. This would recover power and allow a reduced head class pipe downstream of this facility, saving substantial money on the pipe. In-line units have been used for about 15 years by the Yakima-Tieton Irrigation District and have provided reliable service. Cursory investigation revealed that in-line generators with flow and head ranges required for the Sunnyside and Roza deliveries are not in common production. Also, there would be transient problems which would be difficult to estimate, as they would be compounded by other transients from other units on the same pipe system. Therefore in-line generators were not considered at this study level.

Pressure reducing systems using valves or orifices are available from manufacturers in the flow and head ranges required; however, they were not included herein because of reliability concerns with regard to preventing the system pressure getting beyond the valves. If pressure reducing systems were to be used, it would be necessary to insure bypass around them could not be installed. Also, protection from misoperation would be needed to insure the downstream pipe would never see full reservoir head. While not included in this study, in-line generators and pressure reducing systems can be addressed in future studies.

XI. Sunnyside Powerplant and Switchyard

The proposed Sunnyside Powerplant, Bypass and Switchyard are located adjacent to the Sunnyside Canal near its intersection with Konnowock Pass Road. The Sunnyside Powerplant is similar in arrangement to the proposed Black Rock Powerplant arrangement. Details and quantities for the 900 cfs powerplant at the end of the pipeline delivery option (Sunnyside Delivery Option 1) and 1,250 cfs bypass structure were developed but no detailed layout of the 900 cfs powerplant at the end of the canal delivery option (Sunnyside Delivery Option 2) was completed for this study. A comparison of the two turbine units is shown in Table 18.

Concept Description

The Sunnyside Powerplant is an indoor type plant with a structural steel superstructure enclosed with concrete masonry walls. The intermediate and substructure are reinforced concrete. The powerplant consists of a service bay and a single unit bay. A 125-ton overhead traveling crane is provided to handle the powerplant electrical and mechanical items. See Figures 67 through 69.

The Sunnyside Bypass Structure is a separate indoor structure with a reinforced concrete substructure and a structural steel superstructure enclosed with concrete masonry walls. The Bypass Structure houses two 84-inch sleeve valves to dissipate head. Each sleeve valve discharges into a 33-foot-diameter by 20-foot high stainless steel-lined stilling chamber. See Figures 70 and 71. The bypass structure discharges into a 12-foot-diameter steel pipe that discharges into the riprap-lined outlet transition channel that carries powerplant and bypass flows to the Sunnyside Canal.

Table 18. Sunnyside Powerplant Unit Data

Unit Data	Pipeline Delivery Option 1	Canal Delivery Option 2
Number/Type of Units:	1 Francis	1 Francis
Design Discharge:	900 cfs	900 cfs
Design Head:	435 feet	221 feet
Speed:	400 rpm	300 rpm
Assumed Unit Efficiency:	90 percent	90 percent
Power Output at Design Head:	29.5 MW	15 MW
Min. Distributor Submergence (Negative if distributor is above tailwater)	+ 10.6 feet	- 1.1 feet
Max. Scroll Case Dimension	19.4 feet	20.5 feet
Bottom of Draft Tube	El. 859.2	El. 854.5
Turbine Guard Valve:	78-inch spherical	84-inch spherical

The service yard was sized to permit mobile crane access around the structures and a 7-foot chain link fence was provided around the yard for security. Incoming power was assumed to be from a tap on an existing Bonneville Power Authority line about 1 mile southwest of the switchyard. The line tap will include circuit breakers and disconnect switches and a new 69-kilovolt wood-pole line would be constructed from the tap to the switchyard. A 75-foot by 100-foot switchyard is located within the service yard. The switchyard will include transformers, circuit breakers, and disconnect switches. See Figure 66.

Construction Considerations

Canal Bypass: Reclamation assumed there would be a need for a temporary canal bypass during construction. Upstream and downstream earthen cofferdams with geomembrane linings would be constructed in order to connect the transition structures to the canals. Three 9-

foot-diameter corrugated metal pipes between the cofferdams would permit canal operation during construction.

XII. Field Cost Estimates

Field cost estimates were prepared for the major features identified for each option. Field cost estimates include construction contract costs and contingencies. Construction contract costs include itemized pay items and mobilization, plus an allowance for unlisted items. Field cost estimates do not include non-contract distributive-type costs (environmental studies, site investigations, design, construction management, ...) and non-contract corollary-type costs. Field cost estimates do not include land acquisition, relocation, or right-of-way costs that may be required for construction of the project features. Operation, maintenance, and replacement costs are also not included in field cost estimates.

Cost estimates were prepared using available existing design data from past work accomplished by WIS and Reclamation. Aerial topography developed by Reclamation and limited geologic explorations conducted near the proposed damsites were also used to better define features. The amount of data collection is not considered to be at the level required for feasibility-level assessment of project features. Design data collected for future studies can cause future cost estimates to significantly deviate from the cost estimates presented in this report.

Field costs prepared for this study were generated using industry-wide accepted cost estimating methodology, standards, and practices. Major features were broken down into pay items and approximate quantities were calculated for these items based on preliminary general designs and drawings. Unit prices, adjusted for location and current construction cost trends, were determined for the identified pay items.

The appraisal-level field cost estimates developed for this assessment study are for the purpose of evaluating which options should be investigated in greater detail as the Storage Study progresses. **The cost estimates in this report are not intended to be at the feasibility-level required to request project authorization for construction and construction appropriations by Congress.** All field costs are in **June 2004** price level dollars and include mobilization, unlisted items, and contingencies as explained below:

- <u>Mobilization</u> **-** Mobilization costs include mobilizing contractor personnel and equipment to the project site during initial project start-up. The assumed 5 (+/-) percent of the subtotal cost used in the cost estimates contained in this report is based on past experience of similar projects. The mobilization line item is a rounded value per Reclamation rounding criteria which may cause the dollar value to deviate from the actual percentage shown.

• Unlisted Items - Unlisted items are a means to recognize the confidence level in the estimate and the level of detail and knowledge that was used to develop the estimated cost. This line item may be considered as a contingency for minor design changes and also as an allowance to cover minor pay items that have not been itemized, but will have some influence on the total cost. As per Reclamation Cost Estimating Handbook guidelines, the allowance for unlisted items in appraisal estimates should be at least 10 (+/-) percent of the listed items. Typically a value of 15 (+/-) percent is used. Based on the level of detail provided for this study's cost estimates, the unlisted items line item was set at 10 (+/-) percent of the subtotal cost plus mobilization for all features. The unlisted items line item is a rounded value per Reclamation rounding criteria which may cause the dollar value to deviate from the actual percentage shown.

• Contingencies - Contingencies are considered funds to be used after construction starts and not for design changes during project planning. The purpose of contingencies is to identify funds to pay contractors for overruns on quantities, changed site conditions, change orders, etc. As per Reclamation Cost Estimating Handbook guidelines, appraisal-level estimates should have 25 (+/-) percent added for contingencies. Based on the current level of design data, geologic information, and general knowledge of the conditions at the various sites, the contingency line item was set at 25 (+/-) percent of the contract cost for all features. The contingency line item is a rounded value per Reclamation rounding criteria which may cause the dollar value to deviate from the actual percentage shown.

Table 19 is a summary table of the appraisal-level field cost estimates that were prepared for this Assessment. Estimate worksheets showing a detailed breakdown of these field cost estimates are shown in Appendix D. Table 20 shows a comparison of itemized costs (pay items only) for the major features between the Columbia River and MP22.6 of the Roza Canal. Costs shown in Table 20 do not include mobilization, unlisted items, and contingencies. From Table 20, preferred options based on cost can be assembled for the Large Reservoir – Pump Only Option (Option 1), Large Reservoir – Pump-Generating Option (Option 2), and Small Reservoir – Pump Only Option (Option 3). Table 21 compares the combined field cost estimates for each preferred Storage-Pump Option. Tables 20 and 21 do not include costs for the Sunnyside Powerplant and Bypass Structure located at the end of the Sunnyside Delivery System.

Table 19. Summary of Appraisal-Level Field Cost Estimates			
Feature	Large Reservoir	Large Reservoir	Small Reservoir
	Active Storage= 1.3 MAF	Active Storage= 1.3 MAF	Active Storage= 0.8 MAF
	Pump Only	Pump-Generating	Pump Only
	Q (Pump)= 3,500 cfs	Q (Pump)= 3,500 cfs	Q (Pump)= 6,000 cfs
		Q (Generate)= 3,500 cfs	
	(Option 1)	(Option 2)	(Option 3)
Priest Rapids Intake, Pumping Plant, Switchyard,			
and Inflow Conveyance System:			
Discharge 1 (Tunnel/Tunnel)	$620,000,000	$810,000,000	$870,000,000
Discharge 2 (Tunnel/Pipeline)	$860,000,000	Not Priced	Not Priced
Black Rock Dam and Reservoir:			
Dam Type 1: Concrete-faced Rockfill	$1,300,000,000	$1,300,000,000	$1,100,000,000
Dam Type 2: Central Core Rockfill	$1,250,000,000	$1,250,000,000	$1,000,000,000
Dam Type 3: Roller Compacted Concrete	$1,900,000,000	$1,900,000,000	$1,550,000,000
Black Rock Outflow Conveyance System and			
Black Rock Outlet Facility:			
Option 1: 1,500 cfs Power Plant	$590,000,000	$590,000,000	$590,000,000
Option 2: 900 cfs Power Plant	$590,000,000	$590,000,000	$590,000,000
Sunnyside Powerplant and Bypass Structure	$47,000,000	$47,000,000	$47,000,000

Table 20. Cost Comparison of Major Features Between Columbia River and Roza Canal*			
Feature	Large Reservoir	Large Reservoir	Small Reservoir
	Pump Only	Pump-Generating	Pump Only
	Q = 3,500 cfs	Q = 3,500 cfs	Q = 6,000 cfs
	(Option 1)	(Option 2)	(Option 3)
Priest Rapids Fish Screen and Intake	$58,035,920	$64,551,120	$78,815,990
Priest Rapids Pumping Plant	$182,919,070		$275,309,975
Priest Rapids Pump-Generating Plant		$226,254,880	
Black Rock Inlet/Outlet Tower		$85,565,400	
(Priest Rapids to Black Rock Reservoir)			
Inflow Conveyance			
(Priest Rapids to Black Rock Reservoir)			
Discharge 1 (Tunnel/Tunnel)	$186,471,700	$186,471,700	$248,397,600
Discharge 2 (Tunnel/Pipeline)	$357,838,420		
Black Rock Dam:			
Dam Type 1: Concrete-faced Rockfill	$774,496,000	$774,496,000	$621,530,800
Dam Type 2: Central Core Rockfill	$733,280,000	$733,280,000	$573,117,150
Dam Type 3: Roller Compacted Concrete	$1,239,036,300	$1,239,036,300	$980,587,000
Low Level Outlet Works			
For Dam Types 1 and 2	$83,494,115	$83,494,115	$79,000,000
For Dam Type 3	$23,384,515	$23,384,515	$22,000,000
Highway and Utility Relocations	$57,320,000	$57,320,000	$57,320,000
Black Rock Reservoir Outlet Structure	$3,269,850	$3,269,850	$3,269,850
(Black Rock Reservoir to Roza Canal)			
Outflow Conveyance (2,500 cfs)	$306,402,600	$306,402,600	$306,402,600
(Black Rock Reservoir to Roza Canal)			
Black Rock Outlet Facility			
Option 1: 1,500 cfs Powerplant	$104,010,535	$104,010,535	$104,010,535
Option 2: 900 cfs Powerplant	$102,165,985	$102,165,985	$102,165,985

* Mobilization, unlisted items, and contingencies are not included in Table 20.

Table 21. Field Cost Comparison of Preferred Options (Columbia River to MP22.6 of Roza Canal)			
Feature	**Large Reservoir Pump Only Q = 3,500 cfs (Option 1)**	**Large Reservoir Pump-Generating Q = 3,500 cfs (Option 2)**	**Small Reservoir Pump Only Q = 6,000 cfs (Option 3)**
Priest Rapids Fish Screen and Intake	$58,035,920	$64,551,120	$78,815,990
Priest Rapids Pumping Plant	$182,919,070		$275,309,975
Priest Rapids Pump-Generating Plant		$226,254,880	
Inflow Conveyance: Discharge 1 (All Tunnel)	$186,471,700	$186,471,700	$248,397,650
Black Rock Inlet/Outlet Tower		$85,565,400	
Black Rock Dam: Type 2 (Central Core Rockfill)	$733,280,000	$733,280,000	$573,117,150
Low Level Outlet Works	$83,494,115	$83,494,115	$79,000,000
Highway and Utility Relocations	$57,320,000	$57,320,000	$57,320,000
Black Rock Reservoir Outlet Structure	$3,269,850	$3,269,850	$3,269,850
Outflow Conveyance	$303,132,750	$303,132,750	$303,132,750
Black Rock Outlet Facility: Option 1 (1,500 cfs)	$104,010,535	$104,010,535	$104,010,535
Subtotal of Pay Items	**$1,711,933,940**	**$1,847,350,350**	**$1,722,373,900**
Total Mobilization Costs:	**$86,000,000**	**$93,000,000**	**$86,000,000**
PR Intake, PP/PG, Swtchyd & Inflow Conveyance	$21,000,000	$28,000,000	$30,000,000
Black Rock Dam and Reservoir	$44,000,000	$44,000,000	$35,000,000
Black Rock Outflow Conveyance & Outlet Facility	$21,000,000	$21,000,000	$21,000,000
Total Unlisted Items:	**$162,066,060**	**$179,649,650**	**$181,626,100**
PR Intake, PP/PG, Swtchyd & Inflow Conveyance	$41,573,310	$59,156,900	$67,476,385
Black Rock Dam and Reservoir	$81,905,885	$81,905,885	$75,562,850
Black Rock Outflow Conveyance & Outlet Facility	$38,586,865	$38,586,865	$38,586,865
Construction Contract Cost	**$1,960,000,000**	**$2,120,000,000**	**$1,990,000,000**
Total Contingencies:	**$500,000,000**	**$530,000,000**	**$470,000,000**
PR Intake, PP/PG, Swtchyd & Inflow Conveyance	$130,000,000	$160,000,000	$170,000,000
Black Rock Dam and Reservoir	$250,000,000	$250,000,000	$180,000,000
Black Rock Outflow Conveyance & Outlet Facility	$120,000,000	$120,000,000	$120,000,000
Field Cost	**$2,460,000,000**	**$2,650,000,000**	**$2,460,000,000**
Field costs are in June 2004 dollars.			
Field costs do not include Sunnyside Powerplant and delivery systems downstream of MP22.6. See Summary Report.			

XIII. Conclusions

Construction of facilities to pump, store, and deliver Columbia River water to willing exchange participants in the Yakima Basin is technically viable. The following conclusions are based on the technical and cost analyses completed for this Assessment study:

Priest Rapids Intake Facilities

The 3,500 cfs Pumping Plant is the least cost intake and plant facility at Priest Rapids Reservoir. The overall cost of the 3,500 cfs Pump-Generating Option (Option 2) is not significantly greater than the 3,500 cfs Pump Only Option (Option 1); however, operational studies have not been completed for the Pump-Generating Option. These studies may indicate a need to increase plant capacity to ensure annual delivery of exchange water. The decision to provide for pump-generating capability at the Columbia River should be made after these operational studies are complete and costs of the Pump-Generating Option are adjusted to meet the requirements of the operational studies.

Inflow Conveyance System

The cost of the All Tunnel (Discharge 1) inflow conveyance system is significantly less than the cost for the Tunnel/Pipe (Discharge 2) alternative. The Tunnel/Pipe alternative should be removed from further evaluation.

Black Rock Dams

Given the limited available information and explorations of the foundation conditions and construction materials, it is difficult to determine which of the two rockfill dam alternatives would be the optimum choice for the Black Rock site. The relative strengths of both alternatives are that each is an excellent dam to construct in a highly seismic area. The strong and dry downstream rockfill shells provide excellent resistance to extreme seismic shaking. In the unlikely event of a fault offset beneath the dam, neither dam would be expected to fail due to the presence of zone 2 and 3 filters and drains and the “flow-through” capability of the rockfill. In addition, both alternatives feature relatively narrow impermeable barriers, which minimizes expensive and time-consuming foundation treatment work. In short, both alternatives are excellent technical choices, with no clear advantages.

From an economic standpoint, the appraisal-level cost estimates indicate that the central core rockfill alternative has a slight economic advantage. The estimated field costs for the large reservoir are $1.3 billion for the concrete face rockfill and $1.25 billion for the central core rockfill. It should be stressed that appraisal level designs are based on very limited data and explorations, so cost estimates at this stage contain a great deal of uncertainty. More data collection and studies are necessary before fine-tuning the costs to levels that would support cost authorizations and similar efforts.

Due to relative similarity in costs and the equality of technical advantages, it does not appear prudent to rule out either alternative at this point. At the feasibility (or higher) level, additional investigations, explorations, and studies may better define any technical or economic advantages for these two dam types.

The costs of embankment dams are significantly lower than the costs of the roller compacted concrete (RCC) dams. The RCC dams should be removed from further evaluation.

Outflow Conveyance System

Transient analysis of the outflow conveyance system indicates that a surge tank needs to be located close to Black Rock Powerplant to address negative downsurge pressures. To accommodate this requirement, Reclamation moved the alignment of the outflow conveyance from the valley floor to the Yakima Ridge Mountains so that a surge shaft could be excavated near the powerplant. Future studies should investigate alternate alignments assuming power production facilities at the end of the outflow conveyance system are removed.

Black Rock Outlet Facility

The cost difference between the 1,500 cfs and 900 cfs powerplants is small. The bulk of the costs reside with the bypass structure that was conservatively assumed to be the same capacity for both alternatives. The selection of which option to pursue should be made based on economies associated with the Roza and Sunnyside Delivery Systems.

XIV. Recommendations

Should the decision be made to carry the Black Rock alternative into the feasibility design stage, it is recommended that additional data be collected and the preferred options refined for the collected data. Value Engineering methods of analysis should be applied to the identified concepts to identify needs, major cost components, and to reduce overall costs. Value Engineering is a problem-solving methodology that examines component features of a project to determine pertinent functions, governing criteria, and associated costs. Alternative proposals are then developed that meet necessary requirements at lower cost or with an increase in long-term value.

Future Investigations and Studies

General Geologic Investigations

Further geologic study of the reservoir area, damsite, plant sites, and water conveyance alignments will be required during the feasibility stage. Additional geologic investigations will also be required for final design and construction of these facilities. The goal of the exploratory or investigations programs will be to prioritize and produce the amount of data required for that level of study or design. The objective of the engineering geology programs will be adequate identification of all the relevant geologic considerations. Collection and presentation of the geologic data will provide important information regarding geologic and geotechnical considerations for design and construction. Geologic data will be collected to address potential issues relating to stability and strength of the foundation materials, slope stability, deformability of materials, ground-water occurrence and behavior, seepage paths, unwatering and dewatering requirements, groutability, reservoir water-holding capability, seismicity and faulting, reservoir-induced seismicity, landslides, and sedimentation.

<u>Priest Rapids Intake Facilities</u>

<u>Pump-Generating Operational Study</u>: An operational study should be completed to determine how Black Rock Reservoir would be operated if stored water were released back to the Columbia River for power generation purposes.

<u>Pump-Turbine Units</u> : Future studies should investigate the relative costs of a pump-turbine unit installation versus the proposed separate pumps and turbines installation.

<u>Land Development at Priest Rapids Reservoir</u>: According to representatives from the Grant County Public Utility District, the Wanapum Band of the Yakama Nation have jurisdiction over the land proposed for the intake, fish screen, and pumping plant facilities, and they should be consulted regarding construction of these features. See Appendix A.

<u>Inflow Conveyance Systems</u>

<u>Sloping Multi-Level Inlet/Outlet Structure:</u> Future studies should adapt the proposed multi-level inlet/outlet structure to the topography and geology available in the vicinity of the structure. A sloping intake structure should be considered in lieu of the proposed free-standing tower.

<u>Erosion Control During First Filling:</u> Future studies should consider alternative to the proposed erosion protection features required for the initial filling of Black Rock Reservoir.

Black Rock Dams

Use of Asphaltic Concrete Core: Should a central core rockfill prove to have significant technical advantages and impervious soil is not readily available, consideration should be given to a central asphalt core dam. Asphalt simply replaces earthfill as the core material. Such embankments have been used in a number of areas, and have proven to be effective and economical when good quality impervious fill is not an economical option.

Using Crushed Basalt for Filters/Drains: In general, filter and drain materials that consist of subrounded materials (or similar) are preferred to particles with much more angular shapes. However, if such materials are located at a significant distance from the damsite and are thus quite expensive, crushed basalt (available locally) should be investigated as an option.

Grout Curtain Details: A better understanding of the basalt bedrock will likely influence the final design of the grout curtain. Details that would be impacted would include the number of rows (expected to range from one to three), the depth of the holes, and the anticipated grout take.

Potential Adit: Should considerable uncertainty still exist regarding the bedrock permeability even after additional investigations, it may be necessary to construct an adit beneath the water barrier to allow for the possibility of future grouting efforts. For a concrete face rockfill dam, such an adit would be constructed as part of an oversized plinth at the upstream toe. For a central core rockfill, a thick layer of roller compacted concrete (RCC) might be placed beneath the earthfill core on top of the bedrock, and an adit constructed within the RCC. These adits would be sized to allow grouting of additional curtain holes in the event that significant seepage and reservoir losses occur upon first filling of the reservoir.

Highway Relocation: Geologic mapping along the proposed relocated SH24 alignment is not complete at this time and a more detailed geologic analysis is needed in order to determine potential landslide areas and location and depth of bedrock and colluvium layers along the proposed alignment. Given the estimated cost of the road relocation, future studies should consider alternate alignments. Residents of the Black Rock Valley and others have indicated a preference for relocating SH24 north of the proposed Black Rock Reservoir. For purposes of this document it was relocated on the south side of the reservoir because of more desirable topography and the desire to stay outside the Yakima Firing Center Military

Reservation. However, as the Storage Study proceeds further consideration for the relocation of SH 24 should be given and discussions conducted with appropriate State and County entities, Yakima Firing Center Military Reservation, and local residents.

Outflow Conveyance Systems

Future hydraulic and transient studies should refine the flows and pressures associated with the Black Rock and Sunnyside Powerplants. The transient studies should explore the effects of both powerplants shutting down at the same time and one of the plants shutting down while the other one is operating. Future studies should also investigate cost savings associated with removing power production facilities at one or both of these locations. Various highway crossing alternatives near the Black Rock Powerplant should also be evaluated to determine the most cost effective alternative.

Black Rock Outlet Facility

Pressure Reducing Features: The use of pressure reducing features in the Roza and Sunnyside delivery system pipes should be re-evaluated during future studies to see if a fail-safe system can be identified to reduce design pressures in the delivery pipelines.

Future Power Operations Studies : Future power operations studies should be conducted during the feasibility design to better define the rated capacity of the powerplant .

Bypass and Outlet Transition Structure: The design flow for the bypass structure should be re-evaluated during the feasibility study to better match canal delivery requirements and reduce the size of this facility. Alternatives to the outlet channel shared by the bypass and powerplant should also be investigated.

XV. References

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[10] ASCE Manuals and Reports on Engineering Practice No. 79, Steel Penstocks.

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